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AMERICAN WATER WORKS ASSOCIATION

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VOL. 4 JUNE, 1917 No. 2

DECARBONATION AND REMOVAL OF IRON AND MANGANESE FROM GROUND WATERS AT LOWELL, MASSACHUSETTS[1]

By F. A. Barbour

This paper describes the work already done in the removal of iron and manganese from one ground water supply at Lowell, Mass., and the method of decarbonating another supply, recommended to prevent poisoning by the action of carbonic acid on lead service pipes. As of some general interest, a description of the well system from which this city of more than 100,000 people obtains its water will be included; the experimental work on which the methods of treatment were based will be outlined, and the plant which has been constructed for the removal of iron and manganese, with the results of operation to date, will be described.

Previous to 1893 Lowell obtained its water, in great part, directly from the Merrimack River. Typhoid became prevalent and the serious epidemic of 1890–91 making evident the necessity of a better supply, a system of 2-inch and 6-inch wells was driven in the valley of River Meadow Brook in 1892–94. The yield was somewhat less than 5,000,000 gallons daily, or not sufficient for the requirements of the city, and another system of wells, located on the bank of the Merrimack River, was undertaken in 1895. The two systems proved adequate for the demands in point of quantity, but soon the quality of the water became a live issue.

[1] Read before the Richmond convention, May, 1917.

In 1899 the State Board of Health called attention to the effect of the high carbonic acid in the River Meadow Brook wells on lead pipe, stating that some forty cases of poisoning had developed and advising either the removal of the lead services or the abandonment of the supply. Following this condemnation, the supply has been drawn for the most part from the so-called "Boulevard System" on the bank of the Merrimack River, although at times, because of shortage, the wells in the River Meadow Brook valley have been used for short periods.

With the increasing draft from the Boulevard wells, a gradual depreciation in the quality of this supply resulted, due to increasing amounts of iron and manganese and evidenced physically by pronounced color and turbidity in the water as delivered to the consumers. From time to time additional wells were added, the original number of 169 being increased to about 450 in 1913, but despite the effect of the new wells in temporarily reducing the iron content of the water, the amount of this metal increased from 0.22 p.p.m. in 1897 to 1.39 p.p.m. in 1913.

Briefly then, the situation in 1913, when the author was asked to investigate and report on necessary improvements, was as follows: One ground water supply capable of furnishing 4,000,000 gallons daily, condemned because of its action on lead service pipes, but otherwise entirely acceptable, and another so loaded with metallic contents as to render its use diagreeable and unsatisfactory.

As will later appear, the solution recommended for the existing trouble comprehended the treatment of the two ground waters, one for the reduction of the carbonic acid and the other for the removal of the iron and manganese. No surface supply other than the Merrimack River is available within economic distance of Lowell, and a connection with the Wachusett Reservoir of the Metropolitan System, which supplies Boston and adjacent communities, would have cost approximately $2,000,000

The possibility of filtering the Merrimack River involved considerations of some general interest. This river enjoys the distinction of having been made, by special enactment of the Massachusetts legislature in 1878, a free receptacle of sewage, with the result that the pollution is increasing yearly, and is now about four times as great as in 1890. Already at Lawrence, there is a question as to how long it will be practicable to obtain a supply from this stream by filtration without throwing too great a burden on the purifica-

tion plant, and while the good work of the Lawrence filter in reducing typhoid is well known to all engineers, it can be fairly stated that the filtered river water, because of the well-advertised pollution of the stream, has never been attractive to the consumers.

To the people of Lowell, accustomed to the use of a cool, safe supply from the ground, filtered Merrimack River water was out of the question, except as a necessity, and as the experimental work proved the possibility of removing the iron, manganese and carbonic acid at reasonable cost, treatment of the ground water was concluded to be the best method of obtaining a supply not only safe, but attractive.

EXPERIMENTAL WORK IN THE DECARBONATION OF THE RIVER MEADOW BROOK WELL WATER

It has been already stated that in 1899 the water from the River Meadow Brook wells, or the so-called "Cook wells," was condemned because of the action of the contained carbonic acid on lead service pipes. Neutralization or removal of this acid, the first by the addition of lime or soda, and the second by aeration, is the obvious remedy.

An experimental investigation was therefore planned to determine the amount of lead found in the untreated water after passing through 50 feet of ½ inch lead pipe, and also the amount of lead found in the water after passing through similar coils of lead pipe when the carbonic acid had first been neutralized to various degrees by lime or removed by aeration. Three 50-foot coils of lead pipe were used, one for the raw water, another for the lime-treated water and the third for the aerated water. The effect of the period of contact on the amount of lead taken up was determined by taking samples after a period of four hours of standing in or passing through the pipe, and also after a period of fifteen hours.

The apparatus for the addition of lime consisted of a funnel-shaped saturator 24 inches high and 16 inches in diameter at the top. The lime was slaked in a small receptacle at the side of the saturator and introduced as milk of lime by a pipe which led to the bottom of the funnel. Water was introduced from a calibrated nozzle into a pipe which extended vertically through the center of the funnel to the bottom, the supply of water then rising slowly through the milk of lime and overflowing at the top from a circular

weir. The capacity of the saturator was such that when treating 5000 gallons per day of water with 1 grain of lime per gallon, the time of passage of the lime water through the saturator was about one hour.

The flow of lime water was maintained at as nearly a constant quantity as possible, and the amount of raw water was varied when it became necessary to change the quantity of lime added per gallon. The lime water was applied to the raw water without exposing the latter to the air, duplicating, as far as possible, the conditions under which, if this method of removing carbonic acid were adopted, the lime water would be discharged into the suction pipe of the pumps.

In starting the investigation, such portions of the raw, lime-treated and aerated water were passed through the lead pipes during the day that four hours were required between the time of entering and leaving these pipes. At a later date, as always in the case of the fifteen-hour exposure, the four-hour samples were collected from water which had been allowed to stand in the pipes for this length of time. In the interim between the periods during which the water stood in the pipes, the water was passed through at a rate of about 1 gallon in five minutes, the surplus of the treated water being wasted through a bypass. In general, the scheme was intended to represent, as nearly as possible, the actual conditions of a house service pipe.

The regular routine of the experimental station involved the determination, several times each day, of the dissolved oxygen and carbonic acid of the raw and aerated waters, and of the lime-treated water at the time of collecting the four and fifteen hour samples; determinations of the strength of the lime water at two hour intervals, and always, when closing down, for the four and fifteen-hour samples; determinations of the alkalinity, total hardness and iron once each day during August and September, and once each week thereafter; hourly observations of pressure on aerator nozzles; and hourly regulation of the rates of flow.

Table 1 shows the analysis of the raw water as collected from the pumps; Table 2 the relation between carbonic acid and lead content of the untreated and the lime-treated water after remaining in the pipe four hours, and Table 3 the relation between carbonic acid and lead content of untreated and lime-treated water after remaining in the pipe fifteen hours.

TABLE 1

Analyses of raw water as collected from pumps at Cook Well station

DATE	PARTS PER MILLION					PER CENT, SATURATION, DISSOLVED OXYGEN
	Alkalinity	Total hardness	Incrustants	Iron	Carbonic acid	
September 1–6	31.5	57.4	25.9	0.100	39.8	12.4
September 8–13	32.0	53.8	21.2	0.075	39.8	12.6
September 15–20	31.8	55.5	23.7	0.050	43.8	11.7
September 22–27	33.0	57.1	24.1	0.100	46.4	12.3
September 29–October 4	34.0	55.0	21.0	0.035	46.2	10.6
October 5–11	34.0	56.3	22.3	0.100	46.7	8.9
October 12–18	35.0	51.3	16.3	0.050	45.6	8.1
October 19–25					46.5	10.8
October 26–November 1	32.5	56.3	23.7	0.050	47.0	12.2
Average	33.1	55.2	22.1	0.070	44.9	10.9

TABLE 2

Relation between carbonic acid and lead content of untreated and lime treated water after remaining in pipe four hours

DATE	UNTREATED WATER		LIME TREATED WATER			
	Carbonic acid	Lead	Lime added		Carbonic acid	Lead
	p.p.m.	*p.p.m.*	*p.p.m.*	*gr./gal.*	*p.p.m.*	*p.p.m.*
September 2–4	37.5	5.000	16.9	0.99	16.0	2.000
September 9–11	40.5	5.200	18.5	1.08	19.0	1.400
September 17–19	38.0	5.200	16.7	0.98	16.0	2.200
September 23–24	35.5	3.000	16.8	0.98	14.5	1.314
September 27		2.800	17.2	1.00		0.914
October 2–3	31.5	4.000	17.4	1.02	9.0	1.828
October 7	37.5	3.600	17.7	1.04	13.0	1.600
October 10	45.0	5.200	38.0	2.22	1.8	1.400
October 17–19	41.5	6.400	35.6	2.08	0.0	0.400
October 22–23	38.0	4.800	25.8	1.51	6.0	0.714
October 27–28	42.3	5.200	26.1	1.53	0.0	0.257
November 5	39.0	7.000	29.7	1.74	0.0	0.257
November 6–8	41.5	8.000	29.1	1.71	1.8	0.943
November 12	36.0	4.600	29.5	1.73	0.0	0.400
November 13	37.5	5.200	25.5	1.49	0.0	0.800

p.p.m.; parts per million.

It may first be noted that 0.5 part of lead per million has generally been considered to be the permissible maximum in a potable supply. Referring to Table 2, showing the effect of a four-hour period in the pipe, it will be seen that the lead in the untreated water averaged about 5.0 parts per million, or practically ten times the safe limit, and for a fifteen-hour period Table 3 shows 6.7 parts per million, or thirteen times the safe limit. Very obviously the untreated water is entirely unfit for use.

Again referring to these tables, it appears that by the addition of approximately one grain of lime per gallon (100 per cent available) the lead taken up by the water after a period of four hours in the pipe is reduced to 1.5 parts per million, and after a period of fifteen hours to 3.1 parts per million, or respectively three and six times the safe limit. With the increase of lime to $1\frac{3}{4}$ grains per gallon, the carbonic acid was practically neutralized, and the experiments indicated that the corrosive action on the lead was so far reduced that the lead content of the treated water, after remaining in the pipe four or fifteen hours, was approximately 0.5 part per million.

TABLE 3

Relation between carbonic acid and lead content of untreated and lime treated water after remaining in pipe fifteen hours

DATE	UNTREATED WATER		LIME TREATED WATER			
	Carbonic acid	Lead	Lime added		Carbonic acid	Lead
	p.p.m.	*p.p.m.*	*p.p.m.*	*gr./gal.*	*p.p.m.*	*p.p.m.*
August 22	34.5	15.152	17.4	1.02	16.5	6.000
August 27–28	37.2	6.000	18.1	1.06	13.0	2.752
September 2–4	34.8	4.516	17.6	1.03	6.5	5.000
September 9–11	40.6	5.200	18.9	1.11	9.0	1.800
September 17–19	38.0	3.200	16.7	0.98	14.0	1.600
October 2–3	37.0	5.600	17.6	1.03	18.0	1.915
October 7	35.5	6.000	16.0	0.94	11.0	2.600
October 10	36.0	4.344	36.0	2.11	0.0	0.400
October 17–19	39.5	10.000	32.8	1.92	0.0	0.571
October 22–23	34.5	8.000	26.5	1.55	7.2	0.800
October 27–28	34.5	7.200	24.8	1.45	2.6	0.543
November 5	37.5	8.000	30.9	1.80	0.0	0.543
November 6–8	41.0	9.000	29.1	1.70	1.0	0.543
November 12	34.5	5.200	29.9	1.75	0.0	0.257
November 13	35.0	5.600	25.3	1.48	0.0	0.400

p.p.m.; parts per million.

It was concluded from the experiments that 1.5 grains of lime per gallon would sufficiently nullify the effect of the water on lead pipe to make the supply safe, and that a lesser amount would not meet this condition. The cost of such treatment would be small, approximately $2 per million gallons, with reasonable allowance for depreciation on apparatus and for a proportionate charge for attendance by the pumping station force, but this would be a minor factor in the consideration of the expenses to the city incident to this method of decarbonation. The use of 1.5 grains per gallon to reduce the carbonic acid from 45 p.p.m. to 5 p.p.m. and the consequent increase of 45.5 p.p.m. of hardness would entail an additional cost for soap of $455 per million gallons completely softened, and, if it is assumed that in Lowell 1 gallon of the 50 gallons consumed per capita daily would be completely softened, it follows that the removal of forty parts of carbonic acid by lime would result in an increased expenditure for soap of $9.10 per million gallons of water hardened. It was therefore concluded, because of the resulting increase of the total hardness of the water to approximately one hundred parts per million—an amount too great to be satisfactory to consumers accumstomed to soft water—and because of the expense incident to such increase in hardness, that decarbonation by lime was not adapted to the Lowell conditions.

It had been hoped in starting the experiments that the addition of a fraction of a grain of lime per gallon would so far reduce the corrosive effect as to make the supply safe, but the tests indicated that such was not the case. It is of interest to note that the continued passage of water through the lead pipes did not appear to reduce the corrosive effect by the formation of any coating in the pipes.

For the aeration experiments, nozzles with $\frac{1}{4}$-inch orifice, of the type in which a central driving jet collides with four annular, revolving jets just inside the orifice, and a fine conical spray results, were used. It is believed that for a given loss of head this type of nozzle obtains the maximum possible aerating effect.

The aeration experiments showed that with a pressure of 5 pounds on the nozzle the carbonic acid could be reduced from 45 p.p.m. in the raw water to 3.3 p.p.m., and that with this reduction the effect of the water on lead pipes would be so far lessened as to render the supply safe. In stating this conclusion the fact that in the fifteen-hour tests the lead taken up somewhat exceeded 0.5

TABLE 4

Relation between carbonic acid and lead content in aerated water after remaining in pipe for four and fifteen-hour periods at Cook Wells

DATE	FOUR-HOUR SAMPLE		FIFTEEN-HOUR SAMPLE	
	Carbonic acid	Lead	Carbonic acid	Lead
	p.p.m.	*p.p.m.*	*p.p.m.*	*p.p.m.*
September 9–11	3.0	0.114	3.5	0.571
September 17–19	3.5	0.457	3.0	0.543
September 23–24	4.5	0.657	3.5	0.800
October 2–3	3.5	0.457	3.0	0.657
October 7	3.5	0.400	4.5	1.143
October 17–19	4.0	0.400	3.5	0.400
October 22–23	10.0	0.400	13.0	0.743
October 27–28	17.0	0.600	16.0	0.886
October 28	3.0	0.343		
November 6–8	10.0	0.334	9.5	0.943
November 12	22.0	0.943	17.5	1.143
November 13	3.0	0.257		

p.p.m.; parts per million.

p.p.m. is not overlooked, because it is believed that the retention of the water for fifteen hours in the pipe is too severe a test, and the four-hour period of contact more nearly represents the conditions of actual use.

Figure 1 shows the relation between the pressure on the nozzle and the dissolved oxygen and carbonic acid in the aerated water. During the aeration experiments the raw water content averaged 44.9 parts per million of carbonic acid, and from the diagram it appears that with a pressure on the nozzle of 2 pounds, 85 per cent of the carbonic acid was removed; with 4 pounds, 91 per cent; with 6 pounds, 94 per cent, and with 8 pounds 94.8 per cent. To obtain as fine a spray with larger nozzles, additional pressure is required, and from observations of the reduction in carbonic acid effected by $1\frac{1}{4}$-inch nozzles of the aerator in the purification plant at Akron, Ohio, when discharging 100 gallons per minute under 7 pounds pressure, it is estimated that for the same efficiency a 50 to 75 per cent increase in the pressures shown in Figure 1 would be necessary for the size of nozzles used in practical work.

It is of interest to note that for some unexplained reason, the tests indicated that aeration is more effective than lime treatment

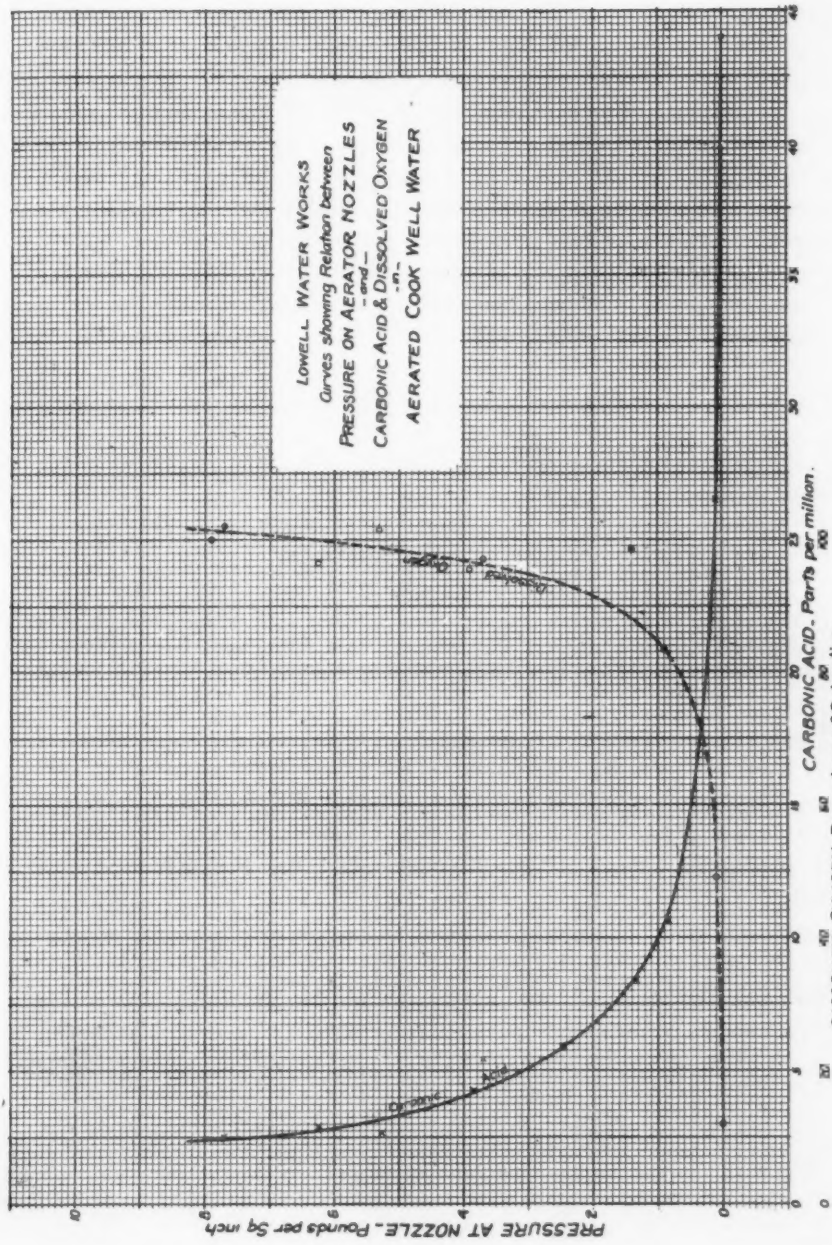

LOWELL WATER WORKS
Curves showing Relation between
PRESSURE ON AERATOR NOZZLES
-and-
CARBONIC ACID & DISSOLVED OXYGEN
-in-
AERATED COOK WELL WATER
PRESSURE AT NOZZLE - Pounds per Sq inch
CARBONIC ACID - Parts per million.
DISSOLVED OXYGEN - Percentage of Saturation
Carbonic Acid
Oxygen

FIG. 1.

in reducing the corrosive action, when an equal amount of carbonic acid is left in the treated water. This result is not what might be expected, because by aeration the dissolved oxygen, which has generally been considered a contributary factor in corrosion, is increased.

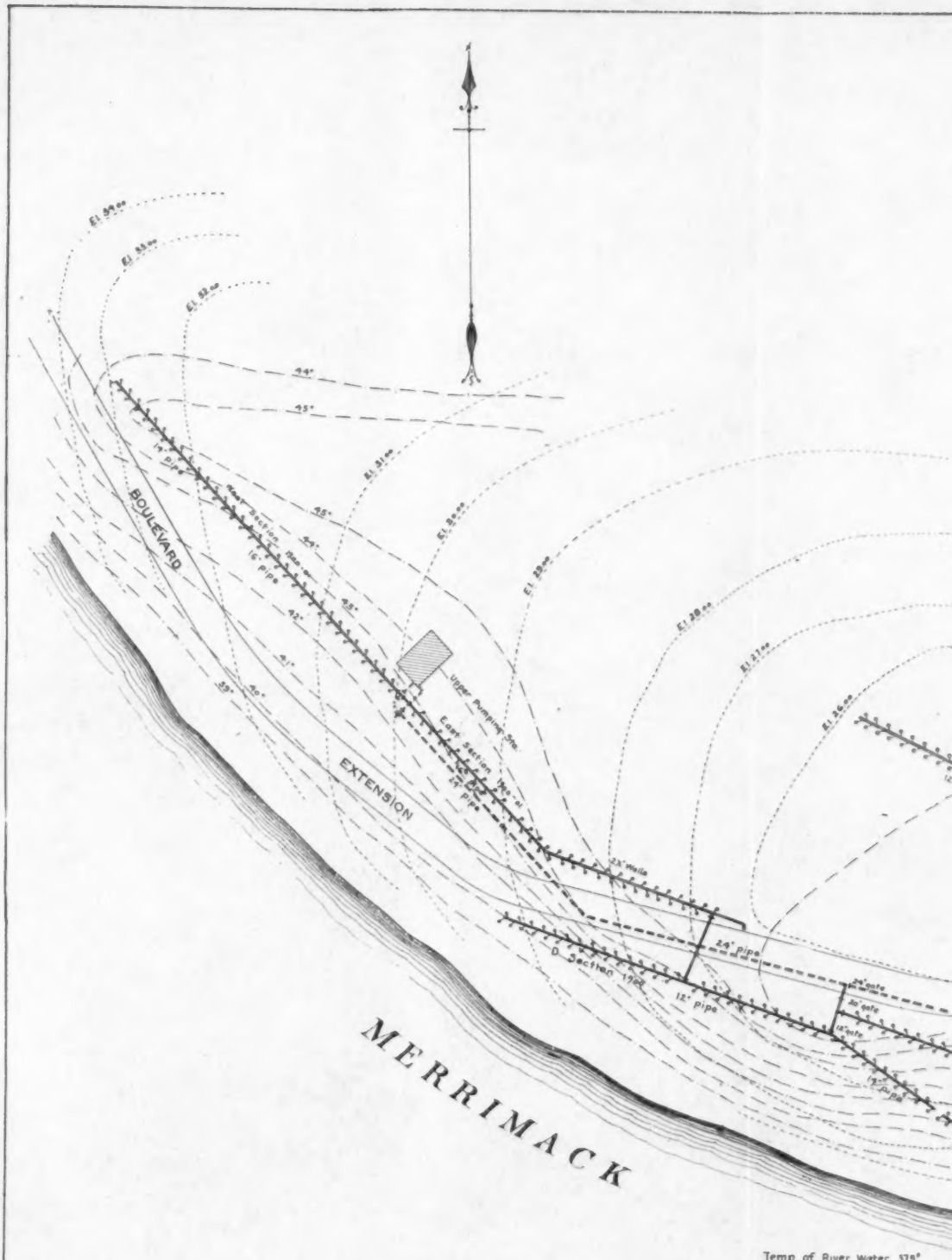
Assuming an additional lift of 30 feet for aeration, the cost of removing the carbonic acid in this way at Lowell, including fuel, interest and depreciation on aerator, collecting well and auxiliary pump, proved to be less than $3 per million gallons. This method of decarbonation was accordingly recommended, and although the plant has not yet been constructed, it will undoubtedly be provided in the near future.

THE BOULEVARD WELL SYSTEM

Before describing the plant constructed for the removal of iron and manganese from the so-called "Boulevard Supply," a brief reference to the system of wells from which some 6,000,000 gallons of water are daily obtained may be of interest.

The well field is a flat terrace on the north bank of the Merrimack River, two miles above the Pawtucket dam, by which the water is held at an elevation of about eight feet below the ground surface. The pooling effect of this dam and the location of the well field on the inside of a bend in the river provide conditions favorable for the deposition of silt and mud. Beneath the surface loam and subsoil, 10 to 20 feet of fine, micaceous, glacial flour is found, and below this, sands of varying size of particle, but all so fine as to make the use of well strainers necessary.

The well system has been gradually extended since the first 169 wells were driven in 1896. In 1900–01, 177 wells and in 1911–12, 118 wells were added, the total number now in use being about 475. Plate I shows the layout of the several lines of wells and also the temperatures and elevations of the ground water from April 6 to 11, 1914. Except for the wells driven in 1911–12, which are located about 700 feet from the river, the distance between the wells and the stream ranges from 150 to 300 feet. A considerable number of wells, driven in 1896 at a less distance from the river, were pulled up and abandoned because of the high iron contents of the water. The theory sometimes advanced that, where metals are dissolved from the underlying soils, the nearer to the stream the wells are located the less the amount of metals dissolved by the water, is not borne out by experience at Lowell.


El. 34.0 m
El. 33 m
El. 32 m
44'
45'
El. 31 m
55'
El. 30 m
El. 29 m
El. 28 m
El. 27 m
El. 26 m
14" Pipe
BOULEVARD
Steel Section 36 in.
16" Pipe
52'
55'
42'
41'
39'
40'
Upper Pumping Sta.
East Section 36 in. cl
24" Pipe
EXTENSION
23' Walls
24" pipe
D Section 1908
12" pipe
24" pipe
20' walls
16" walls
12" Pipe
MERRIMACK
Temp of River Water 37.9°
Elev. - - - 31.0

LOWELL WATER WORKS

BOULEVARD PURIFICATION PLANT

Plan showing

WELL SYSTEM AND LAYOUT OF TREATMENT PLANT

Also

GROUND WATER LEVELS AND TEMPERATURE

APRIL 6-11, 1914

Scale of Feet

F.A. Barbour
Engineer

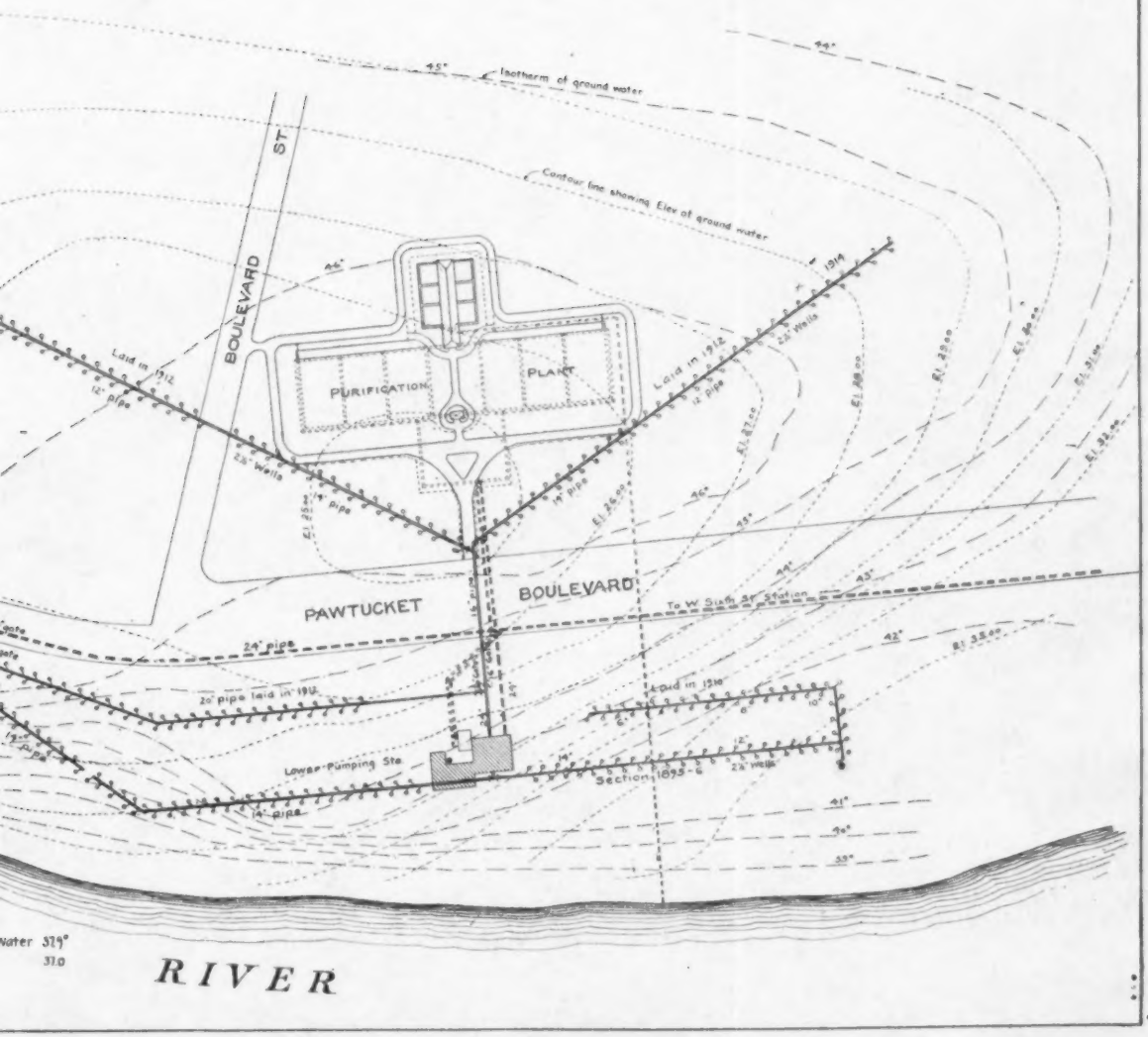


PLATE I

The wells range in depth from 35 to 40 feet, with the points on the average about 25 feet below river level. Extra heavy 2½-inch wrought iron pipe is used, with a bottom section 38 inches long, perforated by 180 ½-inch holes, and with a brass wire soldered into a groove cut spirally, with a ½-inch pitch, to maintain a space of ⅛-inch between the pipe and the surrounding strainer, which is of sheet brass perforated by longitudinal slots, of which there are 20 per inch horizontally and 6 per inch vertically. The corrosion and incrustation of the iron bottom section, particularly in the ½-inch inlet holes, has made apparent the advisability of using brass pipe in this strainer section.

A cast-iron well point 4½ inches in diameter serves to protect the strainer during driving, which is done by heavy drop hammer and without washing. Each well pipe extends to the surface of the ground, where it is capped, a tee being inserted at the level of the suction main, to which it is connected by 2½-inch iron pipe, with a flanged valve in the branch to enable each well to be isolated. The mains are flanged cast-iron pipes laid below the frost line, with grades rising to the air chamber at the pumping station.

Figure 2 shows the color, ammonias and iron in the well water from 1896 to 1916 and also the rate of draft from the wells for the same period. The gradual increase in the iron contents of the water is of interest. Starting with 0.098 p.p.m. in 1896 when the first wells were driven, the iron increased to 0.591 p.p.m. in 1900, falling back to 0.260 in 1901, as a result of 177 new wells driven in 1900–01, and from this point rising to 2.379 p.p.m. in 1911, when it was again reduced to 0.896 in 1912 by the 118 wells provided in this latter year. In the first month after driving the new wells in 1912, the iron in the water pumped from the well field averaged 0.020 p.p.m. and four months later 0.350 p.p.m., these amounts indicating clearly that practically all the water was drawn from the new wells. From an average of 0.816 p.p.m. in 1912 the iron has gradually increased to 2.28 p.p.m. in 1916.

Until the undertaking in August, 1913, of the experimental work in the removal of the iron, no determinations of manganese in the water had been made. It was then found that the amount of this metal exceeded that of the iron, and averaged 2.6 p.p.m. during the experiments. The presence of about 4.5 p.p.m. of iron and manganese, equal to about 200 pounds of metal in the water daily pumped to the city, or 35 tons per year, had naturally led to ex-

tremely unsatisfactory conditions. In solution, when drawn from the ground, these metals, by the oxidation incident to pumping, were, in great part, deposited in the pipe system, the iron nearer the source of supply, and the manganese at more distant points, forming, with the attendant Crenothrix and other fungi, a heavy

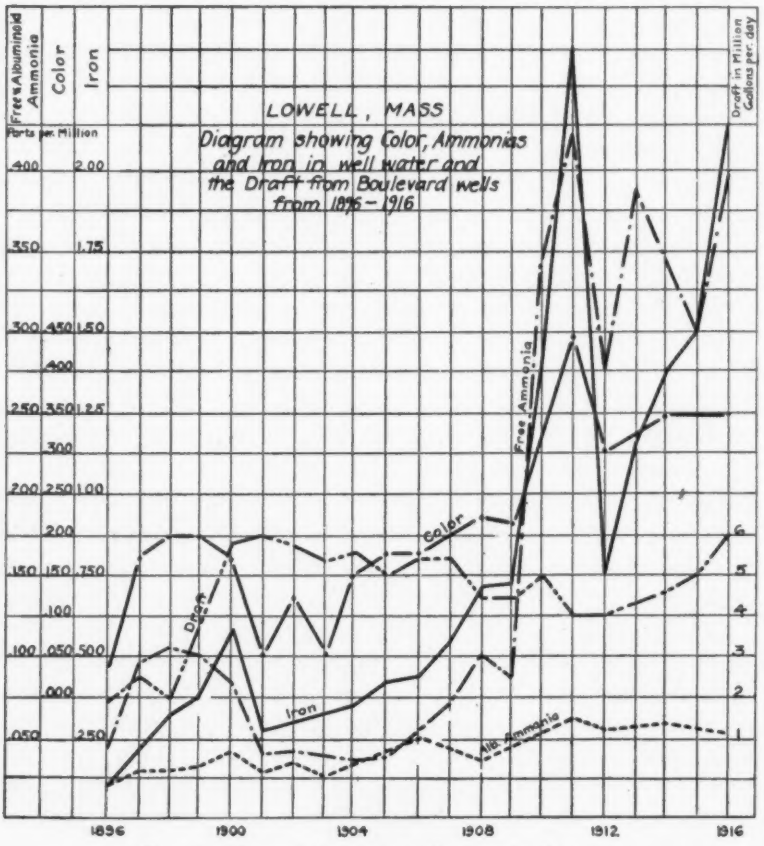


FIG. 2 COLOR, AMMONIAS, IRON AND DRAFT

slime which, when stirred up by unusual drafts, made the water entirely unfit for use.

Iron is present in greater or less amount in practically all sands, and manganese in many sands, from which ground waters are obtained. These metals in fully oxidized condition are practically insoluble, but when reduced by the organic matter in water devoid

of oxygen and containing carbonic acid are readily dissolved. Such a condition is frequently evidenced in the operation of sewage filters, when overloading of the sand and the consequent exhaustion of the dissolved oxygen quickly results in the appearance of iron and Crenothrix in the effluent, only to disappear as soon as the reducing action of the organic matter in the sewage is removed by a reduction in the rate of filtration, or by a period of rest. The determining factor in the amount of metal present in ground water is, therefore, the organic matter in the soil or brought in by the water; and while it may be possible by preliminary soil examination to guard against the location of wells in an area abnormally charged with iron and manganese, the more generally important condition is to limit the rate of draft to that at which the air in the water and soil can oxidize the organic matter present without reducing the metallic oxides to soluble form.

The Boulevard supply is dependent in great measure on infiltration from the river. The amount entering the well field at any point depends, other conditions being equal, on the difference in elevation between the water in the river and the water table at such point. The character of the water which is thus drawn from the river, its deoxidation by accumulated organic matter and its consequent tendency to throw into solution the iron and manganese deposited in the soils, is largely determined by the rate at which the water passes through the soil intervening between the river and the wells. The wells vary in age and in condition of the strainers, which gradually become clogged with silt and metallic deposits. Thus it follows that with no attempt to regulate the vacuum maintained at the station to the condition of the different well lines, the newest wells, or those last cleaned, have furnished the greater part of the water, and observations of the water table have shown that only in the area adjacent to the cleaner wells is the ground water much depressed, a condition illustrated by the lines showing elevation of the ground water in Plate I. The result has been a high rate of infiltration from the river through a limited area, the overloading of the sub-surface sands and the development of the conditions best calculated to throw into solution the iron and manganese in the soils.

By throttling all suction lines connected to the newer or cleaner wells, so as to equalize the supply drawn from the different parts of the field, the rapid clogging of the strainers, due to excessive

drafts, can be materially reduced, and the resulting rate of infiltration through a larger section will at least retard the increase in metallic contents of the water. Also, by extending the well system so as to limit the draft to the rate which can be maintained without too rapid clogging of the stariners or overworking of the adjacent wells, the amount of iron and manganese thrown into solution can be lessened.

It was accordingly recommended in connection with the development of the purification plant, that the well system be extended both up and down stream, that pitometer gaugings of the different suction lines and observations of the elevations of the water table in indicator wells should be made the basis of a system of control by which a more uniform working of the well field might be effected; and that all wells should in future be sunk by wash pipe and the strainer section be made of brass instead of cast iron.

EXPERIMENTAL WORK IN THE REMOVAL OF IRON AND MANGANESE

To provide information for the design of works to remove the iron and manganese from the Boulevard supply, an experimental plant was put in service in August, 1913, and operated for several months under the direction of Clifton L. Rice. This investigation included studies of various combinations of aeration, sedimentation, prefilters of gravel and coke of different depths, and final sand filtration. The regular analytical work consisted of daily determinations of the carbonic acid, dissolved oxygen, iron and manganese in raw water, after aeration, prefilter effluent and sand filter effluent. The prefilter was operated at rates varying from 25,000,000 to 70,000,000 gallons per acre per day, and the sand filter at rates varying from 6,000,000 to 10,000,000 gallons per acre per day.

Different waters require different treatments for the economic removal of metals in solution. When not accompanied by interfering agents, iron can be readily removed by aeration and sand filtration. In the presence of organic matter or manganese, the iron may be held in colloidal form and cannot be so readily precipitated and filtered out. In some waters excessive aeration is possible, and the retention of a certain portion of the carbonic acid is apparently necessary to prevent the organic matter from interfering with the precipitation of the iron.

The experiments therefore comprehended a progressive study,

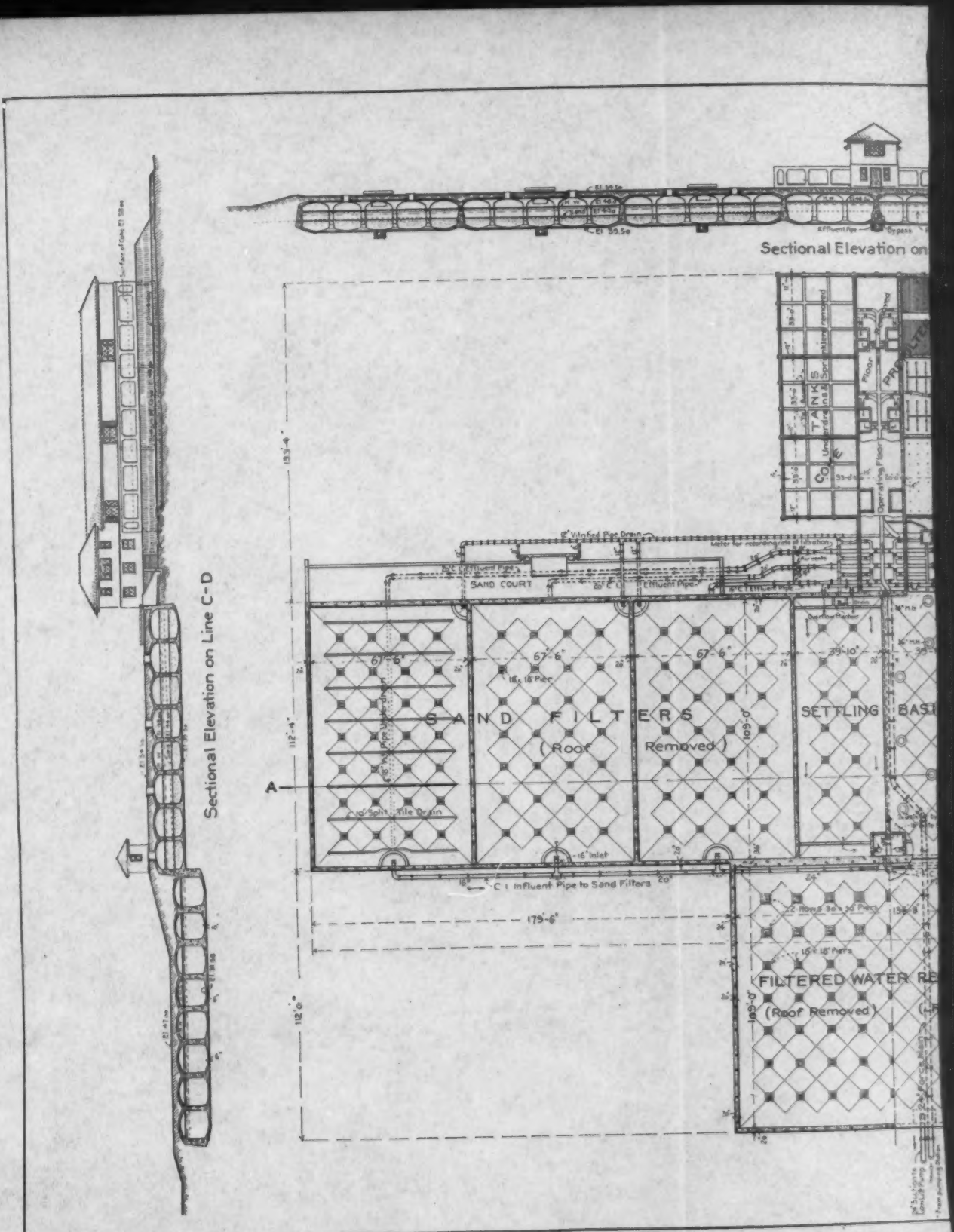
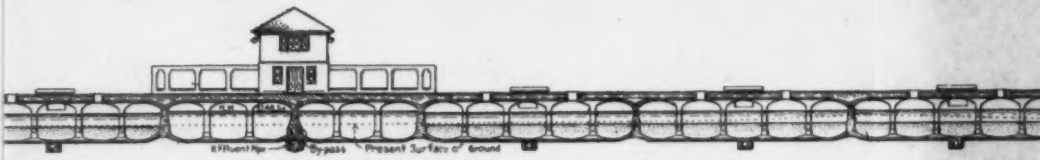
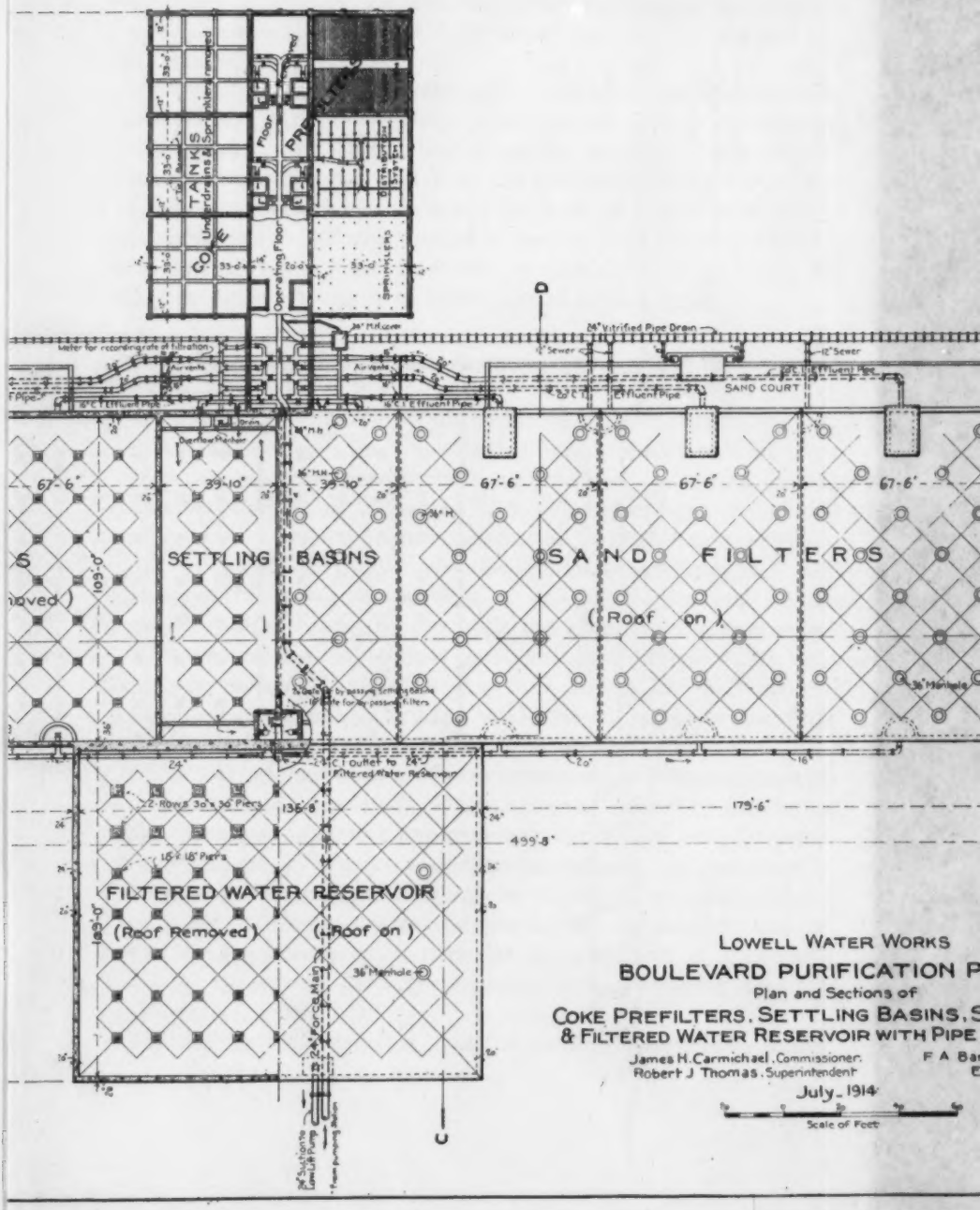

Sectional Elevation on Line C-D
Sectional Elevation on
SAND COURT
SAND FILTERS
(Roof Removed)
SETTLING BASI
FILTERED WATER RE
(Roof Removed)
TANKS
Operating Floor
PRE
A
179'-6"
112'-6"
133'-4"
109'-0"
67'-6"
39'-10"
16" Inlet
C.I. Influent Pipe to Sand Filters

PLATE II



Sectional Elevation on Line A-B



LOWELL WATER WORKS

BOULEVARD PURIFICATION P

Plan and Sections of

COKE PREFILTERS, SETTLING BASINS, S

& FILTERED WATER RESERVOIR WITH PIPE

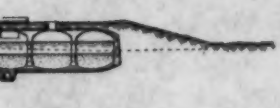
James H. Carmichael, Commissioner.
Robert J. Thomas, Superintendent

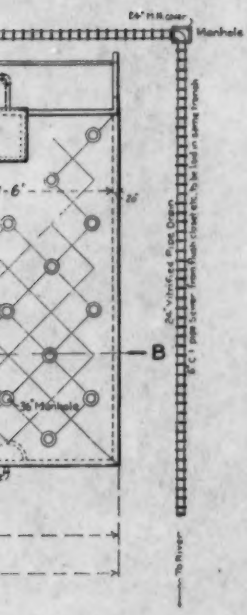
F A Ba

July, 1914

Scale of Feet

PLATE II





WORKS

TION PLANT

of

ASINS, SAND FILTERS

ITH PIPE CONNECTIONS

F A Barbour,
Engineer

starting with simple aeration and leading to more highly developed preliminary treatment by filters of coarse material operated as tricklers or contact beds. It will be sufficient here to state that it was found that the iron and manganese could not be removed by aeration and direct application to sand filters, nor by aeration, sedimentation and sand filters. It was found that excessive aeration was possible, that for the successful removal of the iron and manganese it was apparently necessary to retain 10 p.p.m. of carbonic acid in the prefilter effluent and that the best results were obtained by operating the prefilter as a contact bed rather than as a trickler.

In the final test run with the prefilter, of coke 8 feet deep, operating at a rate of 67,500,000 gallons per acre daily, and the sand filter (30 inches deep, 0.34 mm. effective size, uniformity coefficient 2.56) at a rate of 10,000,000 gallons per acre daily, the iron was reduced from 1.05 p.p.m. to 0.09 p.p.m. per million, the prefilter removing 40 to 60 per cent of the manganese and 55 to 80 per cent of the iron. In this run, with one raking, water equivalent to 590,000,000 gallons per acre passed through the sand filter before it became necessary to scrape.

THE PLANT FOR THE REMOVAL OF IRON AND MANGANESE

Based on the experimental results, a plant with a gross capacity of 10,000,000 gallons daily was designed and constructed, comprising six coke prefilters, 10 feet deep and 0.15 acre in total area, two settling tanks of a total capacity of 500,000 gallons, providing approximately one hour sedimentation, six sand filters with a total area of 1 acre and a filtered water reservoir of 1,000,000 gallons capacity. Plate II shows the general plan and sections.

The plant is located in the well field about 600 feet north of the pumping station. The water is drafted from the wells by one or other of the two 8,000,000-gallon triple, vertical, Holly plunger pumps, and lifted to the treatment plant through a 24-inch force main which extends to and through the prefilter pipe gallery, with branches to each prefilter. The pumps run twenty-four hours daily, at as nearly a regular rate as consumption in the city will permit, and the rate of operating the treatment plant is determined by the discharge of these pumps.

The water is aerated and distributed over the surface of each prefilter by 64 spray nozzles, Figure 3, set 4 feet $1\frac{1}{2}$ inches on cen-

ters and 6 inches above the surface of the coke. The nozzles are formed of brass caps, $\frac{1}{8}$ inch thick, screwed on the top of 2-inch riser pipes, 6 inches long, and perforated by twenty-one $\frac{5}{32}$-inch holes. The cap is domed with a radius of 3 inches and the holes are placed in three circles, 0.21 inch, 0.58 inch and 1.0 inch distant from the axis. It was found by experiment that the coefficient of discharge of the $\frac{5}{32}$-inch holes ranged from 0.80 to 0.85 and that with the holes placed as above described, the sprays, under a head of 2.5 feet at the nozzles, would fall at distances of 7, 16 and 24 inches from the

FIG. 3. PREFILTER AERATING SPRAY NOZZLES

nozzles, rising to heights of about eight-tenths of the head, and increasing the dissolved oxygen content from 12 to 50 per cent of saturation, when discharging at a rate equivalent to 1,250,000 gallons daily on each prefilter, which was the rate adopted for the design of the nozzles.

The prefilters are square reinforced concrete boxes, 33 feet by 33 feet inside horizontal dimensions, set in two parallel rows of three with a space of 20 feet between to provide a pipe gallery (Figs. 4 and 5). The inner walls support the operating gallery substructure, which contains the valve stands and indicating apparatus used in

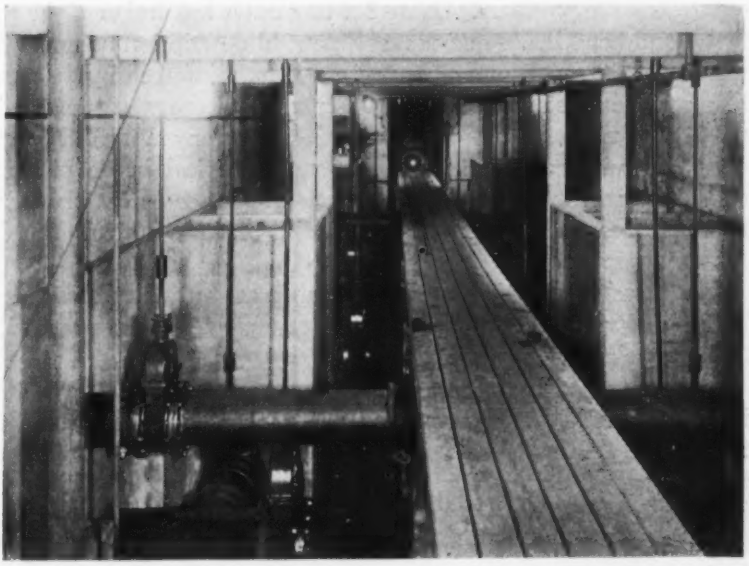


FIG. 4. PREFILTER PIPE GALLERY

FIG. 5. PREFILTER OPERATING GALLERY

controlling the prefilters. Plate III illustrates the design of the prefilters and head house and makes extended description unnecessary.

The prefilter walls are of 1:2:4 concrete, reinforced by deformed bars and designed to withstand full pressure from water level with the top of the wall and with the adjacent filter empty. The wall slabs are reinforced horizontally between vertical inside pilasters, which are reinforced vertically and tied together across the filters by reinforced concrete beams, which also serve to support the distributing piping.

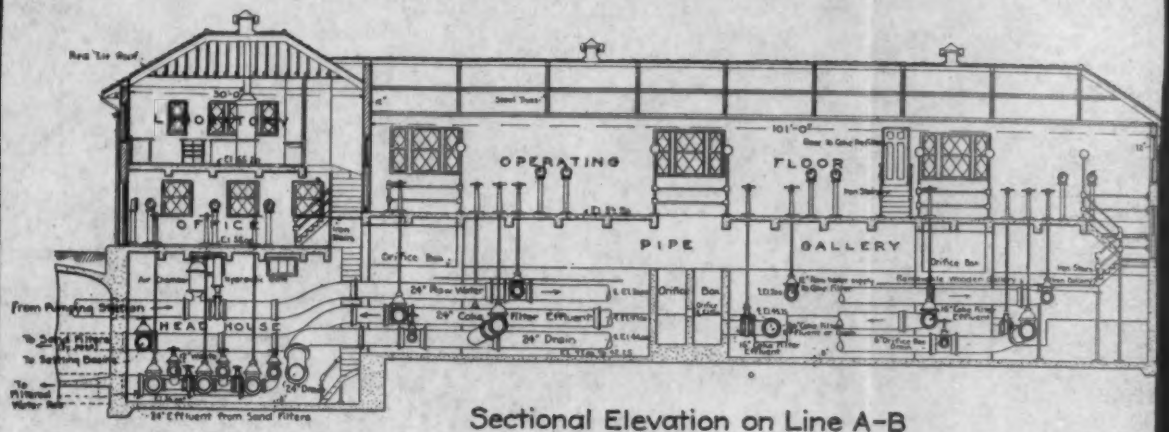
The effluent collecting system is formed of central and transverse concrete channels, covered by slotted reinforced concrete and vitrified clay slabs respectively.

The central collecting channels connect with 20-inch pipes leading to the main drain or by branches to the orifice boxes located in the pipe gallery. In a transverse wall in these boxes there is set a brass plate with an 8-inch circular orifice, and from the difference in elevation between the surface of the water on the two sides of this orifice the rate of operating the prefilters is determined, and indicated by Simplex apparatus located in the operating gallery. The rate is controlled by the valve on the inlet raw water pipe and the depth to which the filter is backflooded by the valve in the effluent branch to the orifice boxes, the elevation of the measuring orifice being such that the filter can be operated to the entire depth as a trickler, if necessary.

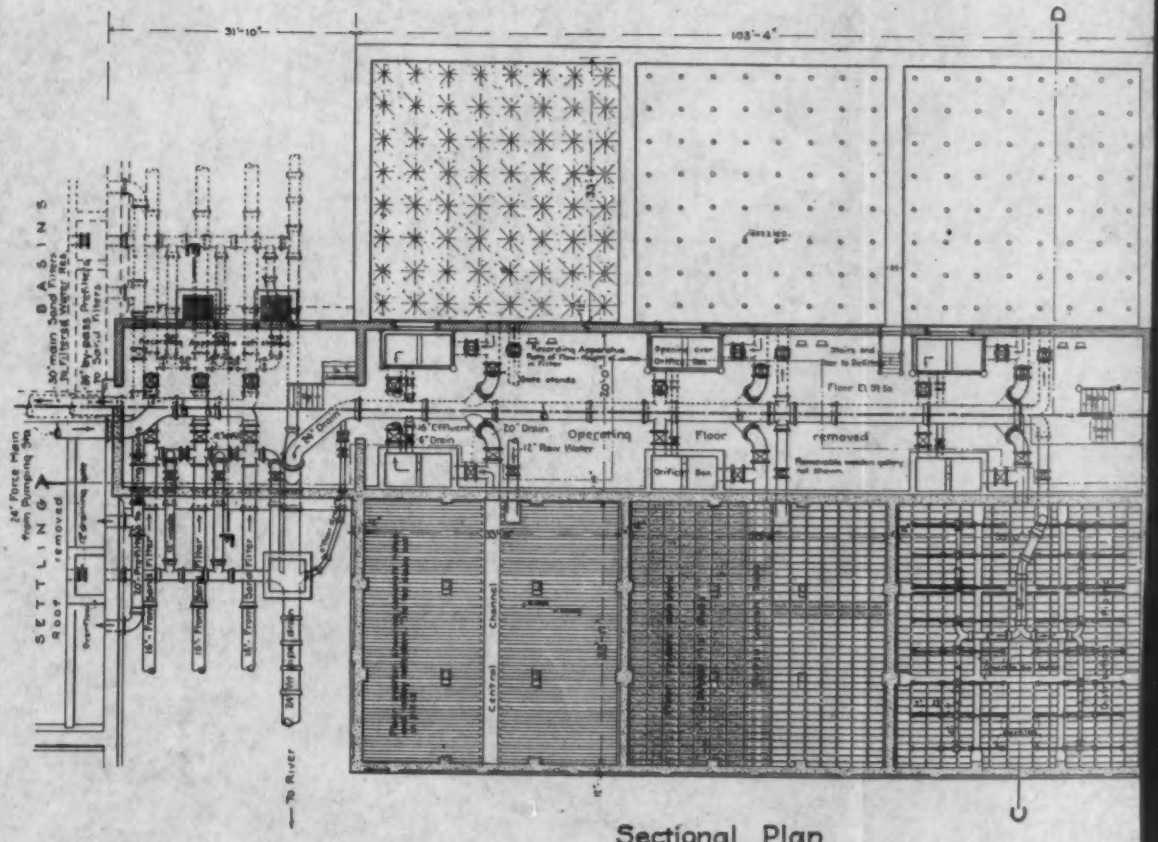
To remove the accumulated deposit from the prefilter coke, the water level is raised to within 1 foot of the top of the wall, the inlet and orifice box branch valves are closed, and the drain connection opened, thus providing an outlet to the river through the 24-inch main drain. In the same trench with this drain a sanitary sewer of cast-iron was laid to prevent contamination of the well field.

The prefiltering material is gas-house coke obtained from the local company, 1 to 3 inches in size of particle and 10 feet deep. The bottom foot was constructed of the larger coke, but above this no separation into sizes was attempted.

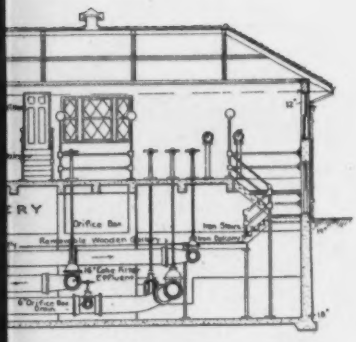
The prefilter effluent flows from the orifice boxes through the substructure of the head house to the settling basins, of which there are two, each of 250,000 gallons capacity. Either or both of these basins can be cut out and the prefilter effluent discharged directly to the sand filters or into the filtered water reservoir. The inlet and outlet ends are baffled by transverse walls, and the floor slopes



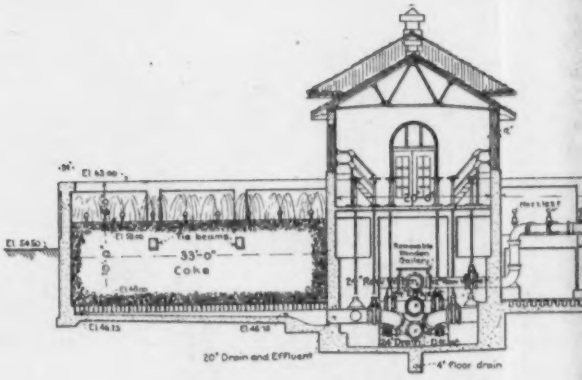
Sectional Elevation on Line A-B



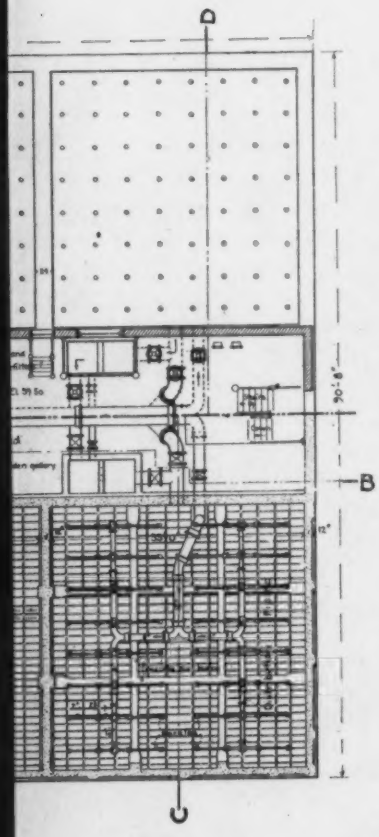
Sectional Plan

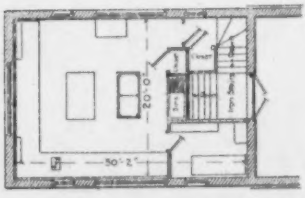


A-B

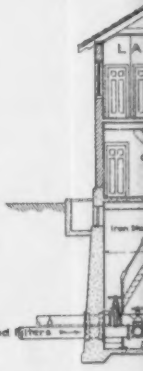


Sectional Elevation on Line C

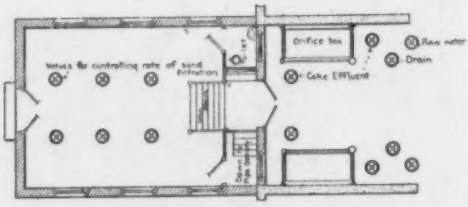




Plan of Laboratory



Sectional Elev



Plan of Office

LOWELL WATE

BOULEVARD PURIF

Plan and Sec

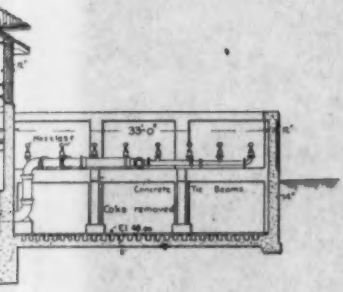
PREFILTERS, OPERA

AND HEAD

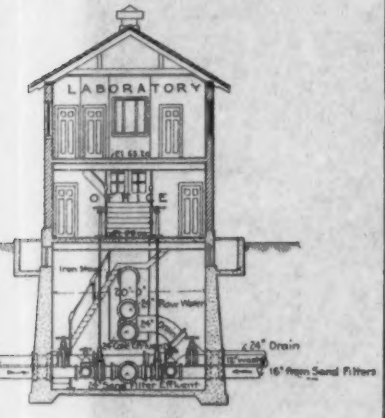
Scale

James H. Carmichael, Commissioner
Robert J. Thomas, Superintendent

July



Line C-D



nal Elevation on Line E-F

LL WATER WORKS

PURIFICATION PLANT

n and Sections of

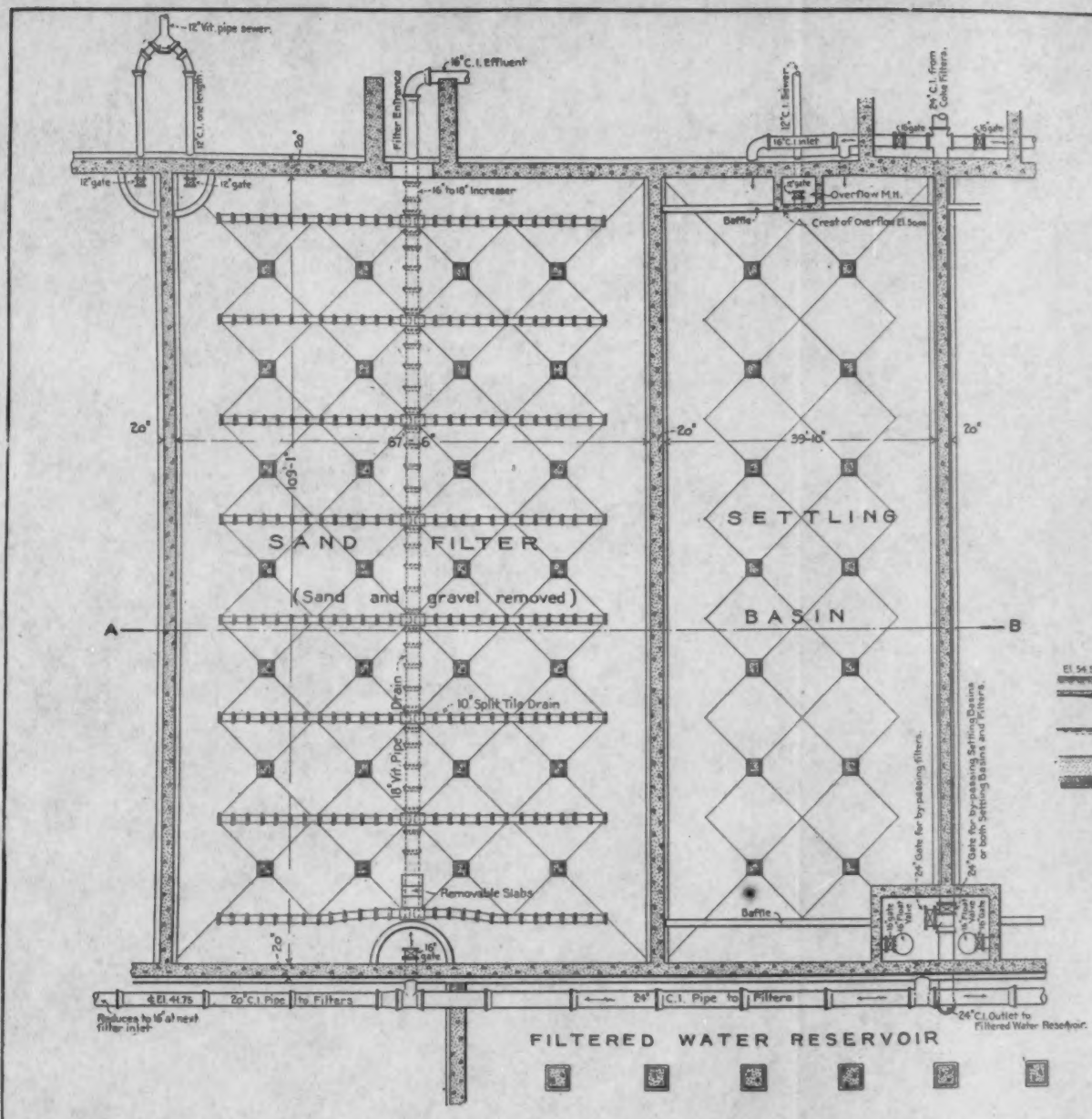
, OPERATING GALLERY

HEAD HOUSE

Scale

F A Barbour,
Engineer

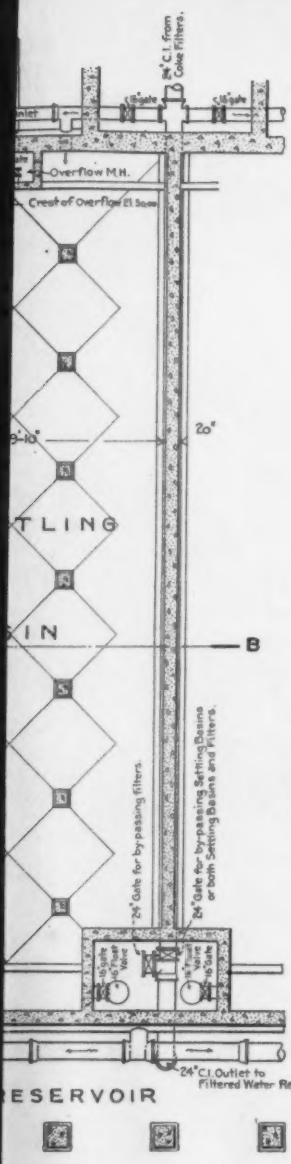
July 1914

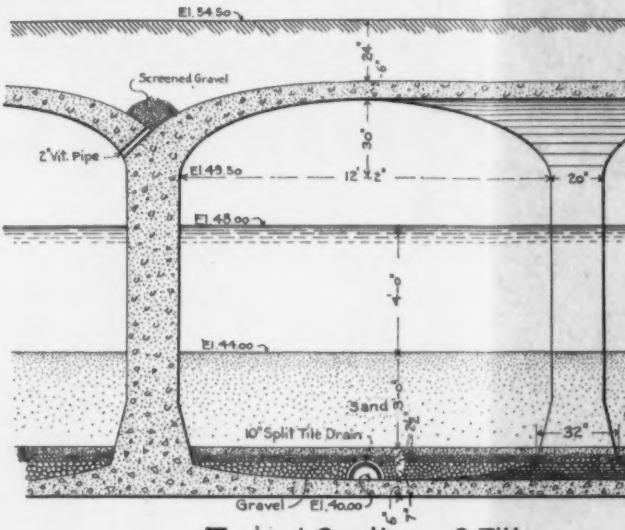


Section on Line A-B

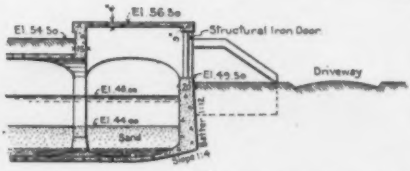
Scale 1 inch = 8 feet.

PLATE IV

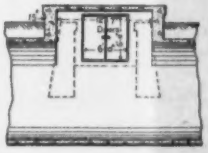




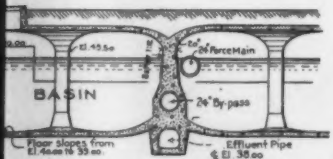
Typical Section of Filter
Scale 1 inch = 2 feet.

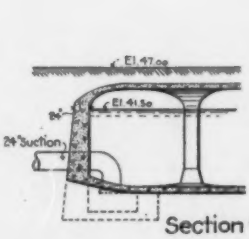


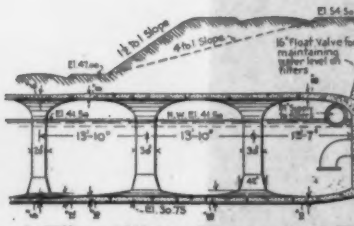
Details at Entrance to Filter
Scale 1 inch = 8 feet.



Detail
Scal







Section through Filtered Water Reservoir at Fl
Scale 1 inch = 8 feet.

LOWELL WATER WORKS

BOULEVARD PURIFICATION PLANT

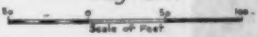
Plan showing details of

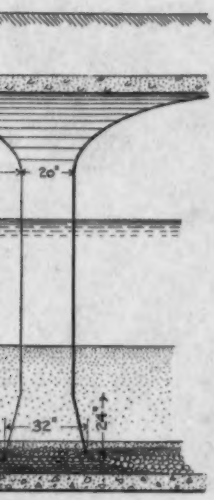
SETTLING BASIN, SAND FILTERS & FILTERED WATE

James H. Carmichael, Commissioner.
Robert J. Thomas, Superintendent.

F. A. Barbour
Engineer

May - 1914.





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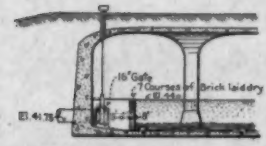
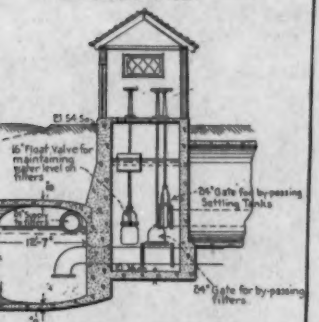


Detail of Filter Inlet.
Scale 1 inch = 8 feet.


El. 54.5
16" Float Valve for maintaining water level on filters
24" Gate for by-passing Setting Tanks
24" Gate for by-passing Filters

voir at Float Chamber

PLANT

ED WATER RESERVOIR

rbour
Engineer

to the inlet end where a connection with the main drain to the river permits discharge of the accumulated deposit.

The settling basins, filters (Plate IV), and filtered water reservoir are of the typical groined roof and floor construction, and demand no special consideration.

The sand filters are six in number, with a total area of 1 acre. From the gate chamber at the outlet end of the settling basins, the water is carried along the south side of the filters with branches to each unit, Plate IV. The filtering material consists of 3 feet of

FIG. 6. SAND FILTER BED EXPOSED FOR CLEANING

sand, having an effective size of 0.35 mm. and a uniformity coefficient 1.60, supported by graded gravel in three layers, the top layer $2\frac{1}{2}$ inches thick, of material passing a $\frac{1}{8}$-inch and held by a $\frac{1}{2}$-inch screen; the middle layer $2\frac{1}{2}$ inches thick, screened between 1-inch and $\frac{3}{8}$-inch, and the bottom layer, which levels up the inverted arch floor, of material passing a 2-inch and held by a 1-inch screen.

The sand cost $1.84 per cubic yard, delivered on top of the filter roof; the gravel was obtained from a bank at South Wilmington, screened, washed and shipped by railroad to Lowell, the cost f.o.b. cars Lowell being $0.66 per ton for the largest size, $1 for the in-

termediate size and $2 for the smallest size. The sand was furnished by a local contractor from a bank 1½ miles from the plant, but on the opposite side of the river. To avoid a haul of some 6 miles by way of the nearest bridge, the material was ingeniously ferried across on a scow, which carried two two-horse carts. A light slack cable, stretched across the river at about water level, served as a guide for grooved wheels, attached to the scow, and the necessary motive power was furnished by two men pulling with about the same motion as in rowing, on a wooden lever with a transverse slot at one end which fitted on the cable with sufficient clearance to permit the lever to be easily slid directly forward along the cable, but which gripped when the pull was applied at the end of the lever.

The effluent of each sand filter unit is carried in a separate pipe to the substructure of the head house, where these pipes are cross-connected so as to permit backfilling of one filter by filtered water from an adjacent unit. In each effluent line there is placed a Venturi meter, and the rate of filtration is indicated and recorded, and the loss of head indicated for each filter unit by Simplex instruments in the office floor of the head house, (Figs. 7 and 8). In this same room are located the stands of the valves controlling the rate of filtration and also instruments indicating the elevation of the water in the settling basins and filtered water reservoir. The general arrangement is such that all operations necessary to the control of the plant can be carried out within the walls of the head house and operating gallery. The upper floor of the head house contains a well-equipped laboratory.

The filtered water, after passing the rate control valves, flows to the filtered water reservoir through a 30-inch conduit formed in the central wall of the settling basins.

From the filtered water reservoir the water is pumped to the central pumping station by turbine-driven, low-lift, centrifugal pumps, installed by the Kerr Turbine Company, two units being provided, one of 7,000,000 and one 10,000,000 gallons capacity. The turbine speed is reduced by helical gears, surface condensers are set in the suction lines, the air pumps are driven by chains from the main shafts and the outboard end of each main shaft carries a small direct-current generator by which the pumping station and purification plant are lighted. A Venturi meter in the force main records the amount of water delivered to the city. Steam for heating the purification plant is piped from the pumping station

FIG. 7. PREFILTERS, OPERATING GALLERY SUPERSTRUCTURE AND HEAD HOUSE

FIG. 8. HEAD HOUSE, FIRST FLOOR

and the lighting current is transmitted through a cable laid in the same trench.

The following tabulation summarizes the results of tests of the turbo-centrifugal pumping units:

	SMALL UNIT	LARGE UNIT
Date of test	June 16, 1916	June 19, 1916
Length of run	2 hours	2 hours
Discharge rate, m.g.d	7,020,000	10,020,000
Total head, feet	20.23	45.22
Steam pressure at throttle, pounds gauge	122.20	122.00
Vacuum, inches, barometer = 29.75	28.48	29.19
Speed pump, r.p.m	640.00	771.00
Total steam, pounds	1,696.20	3,469.50
Moisture, per cent	3.20	1.88
Dry steam, pounds	1,641.90	3,404.30
Dry steam, pounds per hour	820.95	1,702.15
Water h.p	24.95	76.00
Dry steam, pounds per water h.p. hour	32.94	22.40

In determining water horse-power no allowance is made for the head lost through the condenser, nor for work done in driving the air pump. The generators were disconnected during the tests. The efficiencies obtained were about 12 per cent better than the contract guarantees and the apparatus has given entire saitsfaction during the past year, with the exception of the chain drive of the air pump from the main shaft. Such a drive provides the neatest and most economical arrangement for small units, where the exhaust from steam-driven air pumps cannot be utilized, but chains heavier than ordinarily required for the same load and heavy fly wheels or spring sprockets are necessary to overcome the reciprocating action of the air pump and prevent whipping and excessive stretch and wear of the chain.

A brief reference to the effect of cleaning the 9600 feet of 24-inch force main leading from the Boulevard Station to the city may be of interest. This pipe, laid in 1896, had carried for years the metal-laden well water, and the friction loss had become so great that in 1914 cleaning was recommended. Tests in May, 1914, with rates of discharge, measured by pump displacement, varying from 4,500,000 to 6,000,000 gallons per day, made by tapping three mercury columns in the pipe at distances of 2,500 and 3,600 feet apart,

showed values of C in the Hazen and Williams formula ranging from 56.5 to 58.0. The pipe when opened was found to be coated with a heavy, smooth, metallic slime, varying in thickness from $\frac{1}{4}$-inch at the top to $\frac{1}{2}$-inch or more at the bottom of the pipe, but without marked incrustation or roughness of surface. The main was cleaned in September by the National Water Main Cleaning Company and a test made immediately after the cleaning, in the same way as the earlier test, showed for rates of discharge varying from 4,500,000 to 6,000,000 gallons per day, values of C ranging from 124.0 to 127.0. Unfortunately, however, before the purification plant was completed the pipe had rapidly again become dirty and the value of C was found to approximate 85 in November, 1915. The capacity of a soft slime to greatly increase frictional resistance is the interesting condition in the foregoing experience; it is not intended to suggest that where loss of head is due to incrustation the effect of cleaning would so rapidly disappear.

CONSTRUCTION OF PURIFICATION PLANT—COSTS

The purification plant was built partly by city labor and partly by contract, an arrangement which, in a measure, met the demands of local labor leaders but materially added to the cost of the work.

All earth work, the placing of coke, sand and gravel in the filters, and the laying of vitrified clay pipe, was done by city labor; the concrete masonry substructures, filter underdrains and all cast-iron piping were included in a contract which was let to The Charles R. Gow Company of Boston, the lowest bidder.

Work was begun in September, 1914, and practically completed a year later. The contractor was experienced and the mixing and placing of the concrete and the handling of the forms were done economically by a simple but effective plant.

The sand and stone for concrete were hauled from a bank 2000 feet from the work, without screening, the stone ranging from small pebbles to large cobbles. The run of the bank was passed through a crusher, raised by a conveyor to a revolving screen discharging into elevated sand and stone bins, from which by a measuring outlet arrangement the materials were delivered to the mixers in the desired proportions. The mixers discharged into buckets which were moved on small cars running east and west and just north of the masonry structures. From these cars the buckets were

TABLE 5

Cost of Boulevard purification plant, including pumping station extension and pumps

ITEMS	PREFILTERS, EXCLUDING HEAD HOUSE	SETTLING BASINS	SAND FILTERS, HEAD HOUSE, SEWERS AND DRAINS	FILTERED WATER RESERVOIR	PIPE FROM STATION TO FILTRATION PLANT	LOW-LIFT PUMPS AND STATION CHANGES	TOTAL COST
Earth excavation	$1,935.89	$1,681.00	$11,994.68	$8,940.00	$2,058.00	$741.44	$27,351.01
Concrete masonry							
Cement, $1.80 per bbl	3,020.00	2,182.00	10,842.16	3,660.00	53.00	459.00	
Puddle, $4 per cu. yd	19.00	4.00					
Floors, $4.50 per cu. yd		1,180.00	5,570.00	2,120.00		179.00	
Walls, $5.75 per cu. yd		1,662.00	8,025.39	2,540.00	116.00	612.73	
Piers, $6.75 per cu. yd. }		1,735.00	9,360.00	3,360.00		120.00	
Roofs, $6.75 per cu. yd. }							
In prefilters	11,564.06						
In head house		312.00	1,559.00				
Total	14,603.06	7,064.00	35,356.55	11,680.00	169.00	1,470.73	70,343.34
Steel reinforcement							
Bars, 2.5¢ per lb	2,169.33	162.00	758.00	34.00	5.00	25.00	
I beams, 5¢ per lb	426.40		276.10			186.25	
Total	2,595.73	162.00	1,034.10	34.00	5.00	211.25	4,042.08
Coke in prefilters							
Material, $4.75 per ton	4,716.99						
Placing	505.04						
Total	5,222.03						5,222.03
Gravel in sand filters							
Material			4,001.05				
Placing (1200 cu. yds.)			919.41				
Total			4,920.46				4,920.46
Sand in sand filters							
Material			9,439.20				
Placing (5130 cu. yds.)			916.66				
Total			10,355.86				10,355.86
Underdrains, sand filters			960.00				960.00
Prefilters							
Distributors	3,411.27						
Floor slabs	567.60						
Total	3,978.87						3,978.87
Cast iron pipe, etc.							
Pipe, $21.95 per ton	527.00	} 611.00	4,181.40		3,725.42		
Specials, $69.90 per ton	2,710.00						
Valves	2,006.85	445.00	1,227.63		236.00		
M. H. frames, covers, etc		187.00	1,143.70	88.00			
Placing and mis	906 15	146.00	1,496.93		1,262.26		
Total	6,150 00	1,389.00	8,049.66	88.00	5,223.68		20,900.34

Sewers, drains, etc	1,426.72		1,426.72				2,853.44
Recording apparatus	970.41	128.41	2,031.04	34.10		876.69	4,040.65
Pumping machinery						12,182.28	12,182.28
Buildings, etc.							
Structures	8,617.33	1,050.00	4,950.00			2,097.52	
Heating	684.47		394.00				
Lighting	85.00		59.00			586.57	
Laboratory, mis			1,267.10				
Total	9,386.80	1,050.00	6,670.10			2,684.09	19,790.99
Total construction	46,269.51	11,474.41	82,799.17	20,776.10	7,455.68	18,166.48	186,941.35
Administration and mis	562.00	137.00	980.58	247.00	89.00	217.00	2,233.42
Engineering and inspection	2,760.00	685.00	4,917.25	1,245.00	452.00	1,085.00	11,144.25
Total cost	49,591.51	12,296.41	88,697.00	22,268.10	7,996.68	19,468.48	200,319.02
Per million gallons daily capacity	4,959.00	1,230.00	8,870.00	2,227.00	800.00	1,947.00	20,032.00

picked up and dumped by the boom of a stiff-legged derrick, which was supported on the roof of the filters and settling basins and was moved easterly on the central line of these structures as the work progressed.

When water was turned into the plant it was found by test that while the settling basins and filtered water reservoir practically met the contract requirements of a leakage not to exceed $\frac{1}{4}$ inch in depth per twenty-four hours, the sand filters were losing approximately 500,000 gallons daily. As the only structural condition in which the sand filters differed from the settling basins and filtered water reservoir was the main underdrain collector of 18-inch vitrified pipe, surrounded by 6 inches of concrete and laid below the floor of the filter, these drains were first suspected and afterwards proved by test to be the location of the trouble. The filtering material had been placed before opportunity occurred to test the masonry structures for tightness, but by removing the sand and gravel overlying the inspection manholes at either end of the drain, access thereto was possible. Examination showed the pipe to be in good alignment and unbroken, but the joints to be imperfectly filled with cement. The fact that these drains were to be surrounded by concrete had apparently led to lax workmanship and inspection of the joint filling. The contractor, in order to avoid the expense of removing the sand, gravel and filter floor, undertook to stop the leakage by filling the joints in the inside with quick-setting cement, washing with grout, and after drying, painting with asphalt, a long, tedious job, as laborers competent and willing to work in an 18-inch pipe are not plentiful. In this way the leakage, while not altogether stopped, was so far reduced that at the time of the last test the total loss from the six filter units was somewhat less than 50,000 gallons per day, on which basis the work was accepted. The lesson is the necessity of the most careful inspection and testing for tightness of main underdrain collectors before the filter floor is laid.

Table 5 shows the total cost of the purification plant, including day labor and contract work, and also the distribution of the cost, so far as possible, to the different parts of the plant and to the various classes of work. The pay of city laborers ranged from $2 to $2.50 per eight-hour day, with an approximate average of $2.15 per day.

The appended statement of actual cost of the work to the contractor, graciously furnished by Charles R. Gow from accurate material and labor accounts will, it is believed, prove of general value.

OPERATION OF THE PURIFICATION PLANT

Water was first applied to the prefilters in October, 1915, but it was not until July, 1916, that the sand filters were placed in regular service. This was because of the time required to ripen the prefilters, the desire to avoid starting the sand filters in winter without more knowledge of what surface cleaning would involve, and finally because of the delay caused by the necessary repairs to the underdrain system.

TABLE 6

Monthly averages of analyses of water as drawn from wells in 1916

MONTH	DISSOLVED OXYGEN, PERCENT OF SATURATION	PARTS PER MILLION		
		Carbonic acid	Iron	Manganese
January	11.6	22.8	1.64	2.53
February	11.4	21.8	2.20	2.44
March	11.2	22.0	2.34	2.56
April	12.8	22.0	1.64	2.45
May	15.6	21.4	2.28	1.94
June	14.9	20.8	2.35	1.73
July	14.2	20.8	1.92	1.59
August	15.0	21.3	2.28	1.69
September	16.0	21.4	2.49	1.89
October	15.8	21.1	2.51	1.89
November	15.3	21.0	2.28	2.05
December	16.7	20.2	2.09	2.09
Average	14.0	21.4	2.16	2.07

Table 6 shows monthly averages of analyses of the ground water as drawn from the wells during the year 1916. The free ammonia in the well water averaged for the year 0.33 p.p.m., the albuminoid ammonia 0.057 p.p.m. and the total hardness 25 p.p.m.

The prefilters have been operated at an average rate of approximately 50,000,000 gallons per acre daily. The carbonic acid and dissolved oxygen in the water, as applied to the prefilters after aeration, have averaged 14.0 p.p.m. and 52.9 per cent of saturation respectively. The prefilters have been back-flooded to an elevation 1 to 2 feet below the surface of the coke, and the carbonic acid and dissolved oxygen in the effluent have averaged 13.3 parts per million and 54.5 per cent of saturation.

Tables 7 and 8 show the work done by the prefilters and settling basins in the removal of iron and manganese, during 1916.

TABLE 7

Monthly averages of iron in prefilter effluent and settling basin effluent

DATE	IRON—P.P.M.		IRON—PERCENTAGE REMOVED	
	Prefilter effluent	Settling basin effluent	By Prefilters	By prefilters and settling basins
January	0.95	0.89	42.0	45.8
February	0.95	0.84	56.8	61.8
March	0.95	0.75	59.4	68.0
April	0.79	0.58	51.7	64.6
May	0.90	0.57	60.6	75.0
June	1.06	0.58	54.8	75.3
July	0.89	0.76	53.7	60.4
August	1.14	0.84	50.0	63.2
September	1.29	0.98	48.2	60.6
October	1.34	1.00	51.4	60.2
November	1.16	0.88	49.1	61.4
December	1.05	0.86	49.8	58.8
Average	1.04	0.79	52.0	63.4

TABLE 8

Monthly averages of manganese in prefilter effluent and settling basin effluent

DATE	MANGANESE—P.P.M.		MANGANESE—PERCENTAGE REMOVED	
	Prefilter effluent	Settling basin effluent	By prefilters	By prefilters and settling basins
January	1.93	1.85	23.7	26.9
February	1.78	1.70	27.1	30.3
March	1.70	1.62	33.6	36.8
April	1.64	1.57	33.1	35.9
May	1.23	1.15	36.6	40.8
June	1.05	1.00	39.3	42.2
July	0.93	0.82	41.5	48.4
August	0.97	0.88	42.6	47.9
September	0.96	0.92	49.2	54.4
October	1.14	1.08	39.7	42.9
November	1.05	0.95	48.7	53.7
December	1.08	0.96	48.3	54.1
Average	1.29	1.21	37.8	41.6

The removal of 52.0 per cent of the iron and 37.8 per cent of the manganese, or the interception of about 100 pounds of metal daily by the prefilters, makes the unloading of the resulting mass of hydrate an important factor in the control of the plant. This is accomplished by filling the prefilter tank to within 1 foot of the top of the wall and then opening the gate in drain to river. The 4 feet of water over the coke, under average conditions, disappears in 5 minutes or falls 10 inches per minute, providing a velocity through the voids in the coke sufficient to remove all but a fairly constant percentage of the metallic deposit. On the average the prefilters have been flushed once a week during the past year, and the wash water used has equaled 0.80 per cent of the quantity filtered. In addition to the weekly flushing, the upper 8 to 12 inches of the coke has been loosened by forks at intervals of about one month to prevent matting. Experience to date indicates that by this weekly flushing and occasional loosening of the surface layer the deposit of metallic hydrates in the coke can be controlled, and a uniform condition maintained.

The settling basins were cleaned once during 1916, a deposit of about one inch of gelatinous iron and manganese precipitate being flushed into the sewer. From Tables 7 and 8 it will be seen that the settling basins removed only 11 and 4 per cent respectively of the iron and manganese in the raw water. The provision of one or more contact baffles of coarse gravel in each of these basins to assist coagulation and intercept some of the metallic hydrate which now reaches the sand filters has been suggested by the author.

The sand filters have been in continuous service since July, 1916, and have operated at rates varying from 6,000,000 to 7,000,000 gallons per acre daily, depending on the water used by the city. During the year a small experimental sand filter, receiving effluent from prefilter 3, has been run with equally good results at a rate of 10,000,000 gallons per acre daily, and there is no doubt that when the demand makes necessary, a rate of 10,000,000 gallons per acre daily or higher can be maintained in the sand filters.

Table 9 shows monthly averages of iron and manganese in the sand filter effluent from August, 1916, to February, 1917, inclusive.

During the seven months to March, 1917, in which the sand filters have been in regular operation, the iron in the raw water has been reduced from 2.12 p.p.m. to 0.27 p.p.m. and the manganese from 2.14 p.p.m. to 0.03 p.p.m. The water as it leaves the

TABLE 9

Iron and manganese in sand filter effluent

DATE	IRON	MANGANESE
	parts per million	*parts per million*
August, 1916	0.30	0.11
September	0.31	0.01
October	0.29	0.02
November	0.26	0.00
December	0.24	0.02
January, 1917	0.25	0.05
February	0.23	0.03
Average	0.27	0.03

plant is clear and colorless, and experiments indicate that no subsequent precipitate of the residual metallic contents occurs in the water, either cold or hot.

The sand filters, thus far, have been raked when the loss of head, due to the accumulation of surface deposit, reaches 5.0 feet or two-thirds of the maximum possible loss of head. The yield between rakings has ranged from 100,000,000 to 50,000,000 gallons per acre, growing less with each raking and finally necessitating scraping and removal of the surface slime. To March 1, the filters had been scraped twice, about 1 inch of sand being removed each time. The yields between scrapings were 467,000,000 and 412,000,000 gallons per acre, the decreased yield of the second run being due to compacting of the sand by inexperienced workmen rather than of general significance.

The cost of raking has averaged $0.20 per million gallons filtered, of scraping and removing the dirty sand $0.50 per million gallons filtered, or a total of $0.70 per million gallons filtered. No sand has as yet been washed or replaced. These costs are unnecessarily high, due to the inexperience of the workmen and to the fact that adequate pressure for the operation of the ejector is not yet available.

To H. H. Chase, M.Am.Soc.C.E., of the author's office, credit is due for work on the design and general supervision of construction. C. L. Rice was resident engineer during construction and is now chemist in charge of the plant, and to him acknowledgment is also due for valuable assistance rendered.

LOWELL PURIFICATION PLANT, 1914–1915:—COST OF WORK TO CONTRACTOR

Compiled by Charles R. Gow, Contractor

NOTE.—Laborers	$0.25 per hr.	Superintendent	$6.00 per day
Carpenters	0.50 per hr.	Sub-foremen	4.00 per day
Hoisting engineers	0.50 per hr.	Team and driver	5.00 per day

No charge made for personal services of contractor.

General expense items—Superintendent and miscellaneous

(Chargeable to all classes of work in contract)

General labor (including insurance)	$3,830.01
General supplies	1,150.90
Plant depreciation	1,000.00
	$5,980.91 = 8.8 per cent of contract

Cement—11,306.2 barrels

Cement and miscellaneous supplies	$1.281 per barrel		
Labor handling	.053 per barrel		
Teaming	.117 per barrel		
Cost per barrel on work		$1.451	
Cement Shed			
Labor building	$ .007 per barrel		
Materials	.02 per barrel		
Cost per barrel		.027	
Superintendent and miscellaneous, 8.8 per cent on all costs per barrel (see above)		.129	
Total cost of cement per barrel			$1.607

Reinforcing steel—127,245 pounds

Steel (deformed bars)	$ .0157 per pound		
Labor bending and placing	.0040 per pound		
Teaming to job	.0104 per pound		
Total cost of steel in place		.0301	
Superintendent and miscellaneous, 8.8 per cent on all costs		.0027	
Total cost of this item per pound			.0328

Structural steel—17,776 pounds

Steel	$ .0314 per pound		
Labor handling and placing	.0065 per pound		
Teaming to job	.0011 per pound		
Total cost of structural steel per pound		.0390	

Superintendent and miscellaneous, 8.8 per cent on all costs	$ .0034	
Total cost of this item per pound		$0.424

Concrete

General charges—7,828.9 cubic yards total.

(Chargeable to all classes of concrete)

Installing and removing plant

General transportation of plant		
Labor loading	$ .0234 per cubic yard	
Labor unloading	.0157 per cubic yard	
Freight	.0166 per cubic yard	
Teaming	.0772 per cubic yard	
Total for this division per cubic yard	$ .1329	
Erecting, repairing and removing concrete and crusher plant		
Labor	$ .2613 per cubic yard	
Material	.1858 per cubic yard	
Total for this division per cubic yard		.4471
Erecting and removing derricks		
Labor	$ .0779	
Materials and superintendent	.0679	
Total for this division		.1458

Furnishing sand, gravel and crushed stone

Loading and hauling		
Labor loading	$ .3310	
Teaming to crusher pit	.5245	
Total for this division		.8555
Operating crusher and screening plant		
Labor	$ .2262	
Miscellaneous supplies	.0572	
Coal and oil	.2515	
Teaming coal	.0402	
Total for this division		.5751
Total cost of general charge		$2.1564

General forms—7,828.9 cubic yards

(Chargeable to all form work)

Carpenter shop		
Labor	$ .0225 per cubic yard	
Materials	.0353 per cubic yard	
Miscellaneous supplies	.0298 per cubic yard	
Total cost this item per cubic yard		$ .0876

Concrete in filter floors—2,017.6 cubic yards

Not including cement

Forms		
Building		
Labor	$ .0265 per cubic yard	
Materials	.0298 per cubic yard	
Total cost of this division	$ .0563	
Labor erecting	.2489	
General forms (as above)	.0876	
Total cost of forms per cubic yard		$ .3928
Concrete		
Labor mixing	$ .4594	
Labor placing	.8874	
Supplies	.0064	
General concrete (as above)	2.1564	
Total cost of this division per cubic yard		3.5096
Superintendent and miscellaneous, 8.8 per cent on all costs		.3434
Total cost this item per cubic yard		4.25

Concrete in filter walls—2,254.7 cubic yards

Not including cement

Forms		
Building		
Labor	$ .0385 per cubic yard	
Materials	.3407 per cubic yard	
Total cost of this division		$ .3792
Erecting labor	$ 1.4041	
General forms	.0876	
Total cost of forms per cubic yard		$1.8709

Concrete		
Labor mixing	$ .4349	
Labor placing	.6653	
Supplies	.0250	
General concrete	2.1564	
Total cost of this division		$3.2816
Manholes		
Labor	$ .0453	
Materials	.0110	
Total cost of this division		.0563
Superintendent and miscellaneous, 8.8 per cent on all costs		.4584
Total cost this item per cubic yard		$5.67

Concrete in piers—276.8 cubic yards

Not including cement

Forms		
Building		
Labor	$ .4620 per cubic yard	
Materials	1.2234 per cubic yard	
Total cost of this division	$1.6854	
Labor erecting	2.32	
General forms	.0876	
Total cost of forms per cubic yard		$4.0930
Concrete		
Labor mixing	$ .5560	
Labor placing	1.1220	
General concrete	2.1564	
Brick	.2240	
Total cost of this division per cubic yard		4.0584
Superintendent and miscellaneous, 8.8 per cent on all costs		.7173
Total cost this item per cubic yard		$8.87

Concrete in groined arch roof—1,883.2 cubic yards

Not including cement

Forms		
Building		
Labor	$ .307 per cubic yard	
Materials	.4359 per cubic yard	
Total cost of this division	$.7429	

Labor erecting	$1.4062		
General forms	.0876		
Total cost of forms per cubic yard		$2.2367	
Concrete			
Labor mixing	.3322		
Labor placing	.9230		
General concrete	2.1564		
Supplies	.0372		
Total cost of this division per cubic yard		3.4488	
Superintendent and miscellaneous, 8.8 per cent on all costs		.5003	
Total cost of this item per cubic yard			$6.19

Prefilter concrete—1,396.6 cubic yards

Not including cement

Forms			
Labor erecting	$2.1326 per cubic yard		
Materials	.9780 per cubic yard		
General forms	.0876 per cubic yard		
Total cost of forms per cubic yard		$3.1982	
Concrete			
Labor mixing	$ .3114		
Labor placing	1.4514		
General concrete	2.1564		
Total cost of this division per cubic yard		3.9192	
Special plant			
Tower labor, Erecting	$ .2425		
Tower materials	.0663		
Rental of hoist engine	.1790		
Total cost of this division per cubic yard		.4878	
Tile slabs			
Material	$ .3297		
Labor placing	.09		
Teaming	.0257		
Total cost of this division per cubic yard		.4454	
Waterproofing—Saum's preservative		.1575	
Superintendent and miscellaneous, 8.8 per cent on all costs		.7223	
Total cost this item per cubic yard			$8.93

Underdrains in six sand filters—3,192 linear feet

	Per linear foot	Per filter
Pipe	$ .2023	$107.63
Labor and teams	.0924	49.17
Total cost of pipe and labor	$ .2947	$156.80
Superintendent and miscellaneous, 8.8 per cent on all costs	.0293	13.80
Total cost of this item	$ .3240	$170.60

Cast iron pipe—338.6 tons

Labor laying and caulking		$5.305 per ton	
Teaming		.437 per ton	
Erecting trolley			
Labor	$ .071		
Materials	.082	.153	
Total for this division per ton			$5.895
Superintendent and miscellaneous, 8.8 per cent on all costs			.519
Total cost this item per ton			$6.41

EXPERIENCE WITH A SUBMERGED PIPE LINE IN THE PUERTO BARRIOS WATER WORKS[1]

By T. Howard Barnes

Puerto Barrios is the Atlantic seaport of Guatemala, the most populous of the Central American republics. For many years the rich interior sought a railroad connection to this outlet, finally realizing it in 1908. The president, Manuel Estrada Cabrera, recognized that the strip of sand-edged swamp called Barrios was destined to become important, and at the inauguration of the railroad in January, 1908, solicited a proposition for the complete sanitation of the port, that it might become in appearance, convenience and healthfulness, worthy of its importance as the eastern gateway of the republic. A substantial appropriation for the necessary studies and for provisional cleaning, ditching and other works of sanitation was made. The writer was designated to prepare the report and estimate of cost.

The report presented in the spring of 1909, called for about 3,000,000 cubic yards of filling, a sea wall, drains, sewers, water system, pavements and parks. The total cost proved too great for immediate execution. At the date mentioned, the terminal comprised a wharf, railroad station, round house, hotel, custom house, government house and a few scattered wooden residences, with a beach line of thatched cabins. The water supply depended upon rain-water cisterns and a 2-inch pipe well deriving water from a stratum of coarse sand 100 feet below the surface at a point about half a mile inland.

Natural growth of traffic and population brought the place face to face with a water famine, forcing the railroad company to introduce an adequate supply. The most available source was a mountain stream 7½ miles away, as measured by a practicable land line, or 5½ miles by a route in part across an arm of the bay. The submerged portion on the latter route was about 15,000 feet in length. The water attained a maximum depth of 24 feet. The bottom was

[1] Read at the annual convention at Richmond, May, 1917.

known to be of ooze. The land route involved several miles of swamp and rugged rock trench. The sea route led to fairly easy land connections.

The all-land route involved an expense greater than seemed justified, while the cost of the shorter route would likewise be formidable should the submerged portion be laid with the ball-and-socket type of pipe joint. The size of the pipe demanded by the governing factors of quantity and pressure was 8 inches in diameter. The shorter route was chosen after investigation showed the Universal joint was adapted for continuous submerged pipe laying.

This joint, as is well known, is an iron-to-iron joint, only white or red lead being used in making it up. The pipe lengths are 6 feet. There are bulbous rings and ear lugs near the ends, which ends are machined in a lathe to certain exact tapers. The female taper is about $2\frac{1}{2}$ degrees and the male slightly less. The pipes are held by bolts passing through the ears. There are two at each joint on pipe up to 16 inches in diameter, except that four are used for 12-inch and 16-inch pipe made for pressures exceeding 175 pounds per square inch. With two bolts a flexibility is obtained, when the tension on the bolts is moderate, in the plane vertical to the line passing through the two ears on any pipe. That is, a vertical movement is possible when the pipe is laid with ears in a horizontal position.

Tests were made regarding the extent of the flexibility and the behavior of the pipe when under water pressure. Eight 6-inch pipes were jointed and blocked up in line. Water was admitted and put under a pressure of 150 to 200 pounds per square inch. The blocking was removed in the middle, permitting a deflection of 8 inches in a five-pipe length, as measured from a chord at the center of bend. No resulting effect in loss of pressure ensued. The pipe was then deflected at the end piece until 4 inches was attained from a tangent extended from the adjacent piece, no leak ensuing. Slackening of bolts was tried, resulting in finding that a withdrawal of $\frac{1}{8}$ inch produced a leakage of about two drops per second. These tests decided the use of this pipe and the adoption of the short route across the Bay. The same pipe was also adopted for the land portion of the line.

The method of laying was as follows: Two small deck lighters or floats were strapped side by side with about 6 feet clear space, forming a catamaran. A trussed trough tailing into a pair of long planks, and having a sliding way for paying off the pipe was sus-

CATAMARAN EMPLOYED IN LAYING SUBMERGED PIPE

PIPE ON THE WAYS OF THE CATAMARAN

pended near its upper end from a Jennywink or light framed derrick on one of the lighters. To the cradle were attached sufficient barrel floats to make it nearly buoyant when supporting the pipe.

The length of the cradle with its tail planks was finally made 100 feet, sufficient to reach the bottom at a slope of about 14 degrees. This length was at first not so great, but later was made so.

HELMET USED IN DIVING

The pipes lay net 6 feet. For submerged use they were tested again on the ground, eliminating about 40 pieces out of the 2400 laid under the water, or 1 in 60. The joints were smeared with red lead. The ears, of course, were kept in a horizontal position, so as to permit vertical deflection in the pipe. The bolts were enveloped in a mixture of crude oil and cement held in place by muslin.

The use of red lead was adopted on the joints for subaqueous work. It has been found that red lead adheres better than white

for wet work. For earth trenches white lead was used. The bolts on land were protected by lean cement-sand mortar and bound in muslin. This was because the trenches were shallow and organic material was likely to rust the bolts rapidly.

The position of the catamaran was manipulated and maintained by six lines, two each forward, aft and abreast, while alignment was maintained by tall ranges on shore. Progress was quite rapid as a rule. To guard against accident from squalls, the cradle and pipe were lowered each night and noon, whenever the work was suspended. Work began August 24, 1916, and the last pipe was laid October 19, 1916. The number of working days was 47 in laying 14,400 feet of pipe. The average per day was 306 feet. The biggest day's work was 600 feet, and for one week 2982 feet.

At intervals of about 100 pieces, a water test was applied by means of a hand pump. When about 7000 feet had been laid the test failed to show a pressure on the guage. Air was then applied and two or three issues located. Work was not arrested, but a diver was sent for to make an examination. While waiting for the arrival of the diver, the pipe laying was finished.

The air issues indicated accurately the situation of the broken pipes. There proved to be seven in all. Of these two were in pipe split lengthwise and five where there were annular cracks at the shoulder of the toolwork against the rough iron. These fractures were probably due to too much deflection in tailing off from the cradle to the bottom in the case of pipe laid before the cradle was lengthened. It is also probable that the sea chop at times produced a compression on the side of some joints at the tail of the cradle.

It was demonstrated in the repairs that single pipes were easily removed. The oil cement mortar for protecting the bolts against rusting rendered the removal easy and lifting two pieces each way from the defective one sufficed to liberate the latter and admit the new pipe.

The experience herein recounted of the use of this joint indicates a possibility in laying submerged lines with greater economy than has been thought possible for similar situations. It is believed that it is possible to lay this pipe in certain submerged situations up to the limit of the sizes where only a single pair of ears are required.

Attention should be called to the favorable circumstances obtaining as to bedding of the pipe in this instance. The sea bottom is a

pure bed of ooze into which the pipe sank for nearly its full diameter in the process of laying. The bed is, therefore, one ideally prepared by nature. The lightness of the pipe lengths for transporting was an important factor. The facility for jointing in this region of crude help and adverse weather was also a favorable circumstance. This feature was likewise appreciated in the land portions, where swamp and declivity presented difficulties in handling heavy pieces.

It has been mentioned that several pipes split along the barrel and had to be removed after being inserted and tested. The experience gained in using this pipe, both here and in other foreign locations, where several handlings are entailed by ship's slings, etc., leads the author to believe that the cost of such testing is justified. The cost amounted in this case to about 1.6 cents per running foot on 15,000 feet of 8-inch pipe, which sum is not a serious factor. The author would not be understood as favoring greater caution in regard to this pipe than with any other manufactured for use in similar circumstances. The handling by ship's slings is always a severe test on any fragile material, and the fact that the smaller weight of individual pieces permits handling them in nets means that they receive a rather heavy hammering.

Among the illustrations is one showing the simple head piece which constituted the diver's equipment. A negro laborer on the work was broken in to make the repairs, but the foreman in charge himself inspected each replaced joint.

The final air test was 60 pounds, initial pressure, which in 24 hours reduced to 40 pounds.

The cost of the labor on the 14,400 foot submerged portion of the work is approximately given in Table 1.

TABLE 1

Cost of laying 14,400 feet of submerged pipe

	COST	COST PER FOOT
Pipe testing, handling and laying	$1,600.00	$0.1111
Hire of launch	320.00	.0222
Building cradle	100.00	.0069
Repairs and preparation of catamaran	150.00	.0104
Storing of plant after finishing job	150.00	.0104
Replacing defective pipe, line tests, etc., (includes about $350 expenses for diver and helper sent from the United States but not retained)	900.00	.0625
Total	$3,220.00	$0.2235

The extraordinary amount for repairs is illustrative of the expense due to delays in getting together special appliances when work is conducted in remote situations. In the total not only was the diver's expense a large item, but air pumps and power for testing out was considerable, and overhead expense was, of course, a continuing item.

The author was not present during the construction and the work was done under the direction of Alfred Clark, general manager of the railroad company. Carl Riggs was in general charge and constructed the land line; William J. Riggs had immediate charge of the submerged line and inspected all the joint repairs, using the head helmet; the successful completion of the work is largely due to them. Charles Schubel represented the Central Foundry Company.

ULTRA VIOLET RAYS; THEIR ADVANTAGES AND DISADVANTAGES IN THE PURIFICATION OF DRINKING WATER[1]

By R. R. Spencer[2]

The purification of drinking water on a large scale has always been a problem of greatest moment to sanitarians and public hygienists. None of the present accepted methods has a general application, but in each case an appropriate method must be chosen which can be operated efficiently and economically. In the application of ultra violet rays to the purification of water the limitations of the method must be particularly considered. On the other hand, in view of the well established bactericidal property of these rays and the results of certain tests, hereinafter tabulated, on two types of ultra violet ray apparatus at the laboratory of the Sanitary District of the Great Lakes, it is believed that this method of purifying drinking water will give satisfactory results, under suitable conditions.

Properties of ultra violet rays. The ultra violet rays, which are present to a greater or less extent in all light, are invisible rays, lying beyond the violet end of the spectrum.

In 1877, Downes and Blunt (1) showed that sunlight was an efficient bactericidal agent. In 1892 Marshall Ward (2) by analyzing the effect of the spectrum thrown on infected agar plates demonstrated that the bactericidal action "begins at the blue end of the green and rises to a maximum as we pass to the violet end of the blue." He further observed that the bactericidal activity extended far into the ultra violet when the light passed only through quartz. Following these observations considerable attention was given to the physiological effect of light. The ordinary phenomenon of sunburn was shown to be due to the actinic or chemical rays and not to the caloric rays, as was formerly thought. Finsen (3) has obtained very excellent results by using the chemical rays of light in the treatment of skin diseases.

[1] Read before the Illinois Section at Urbana on March 13, 1917.

[2] Assistant Surgeon, U. S. Public Health Service.

Of all the spectral rays the ultra violet is the most refrangible; at the same time it possesses the strongest chemical effect and the least heating effect. The other extremity of the spectrum presents the opposite phenomenon. The red and infra-red are the least refrangible and the chemical effect is at a minimum. The visible rays of the spectrum are those that have wave lengths between 0.76 micron and 0.4 micron. Both the ultra violet and the infra-red rays lie outside of these limits of visibility (infra-red wave lengths are between 300 and 0.76 microns; ultra violet rays are between 0.4 and 0.1 micron). The ease with which these rays of short-wave lengths are absorbed is quite characteristic. Ultra violet rays are absorbed by nearly all material, except fused rock crystal (quartz), fluor spar, a specially prepared glass called Uviol, air and water. Even these interrupt part of the rays; for example, several centimeters of air absorb completely the wave lengths below 0.185 micron while quartz absorbs the wave lengths below 0.2 micron. Besides their bactericidal or abiotic power, as it has been called, ultra violet rays are capable of accelerating many chemical reactions, and in this instance apparently act as a catalytic agent. For example, chlorine and hydrogen, which combine slowly in diffuse light, combine with an explosion under the influence of ultra violet rays. White phosphorus is changed into the amorphous form (red phosphorus). Ozone is produced in air. Hydrogen peroxide is formed in clear water. The blackening of photographic paper is due almost entirely to the chemical rays. In handling this material it is exposed only to a red light, which holds back the shorter waves and allows only red and infra-red to pass. Commercially the ultra violet rays are used to detect willemite ($2ZnO$, SiO_2), a zinc ore in furnace tailings (4). This substance has a peculiar luminous glow when exposed to the ultra violet rays. Further, it is claimed ultra violet rays have the power of bleaching linseed oil, ripening the pods of vanilla beans, destroying or attenuating the action of ferments, and hydrolysing saccharose.

Measurement of rays. Up to this time no satisfactory method has been devised by which ultra violet light can accurately be measured. Since the rays are invisible the amount of radiation can only be estimated by noting their physical or chemical effect on various substances. The amount of fluorescence produced on certain compounds, the speed with which photographic paper is blackened and the decomposition of various chemicals are all methods which

may be employed for determining the ultra violet "candle power" of a lamp. Probably the most reliable method is the determination of the bactericidal power of a lamp. Von Recklinghausen (5) has recommended that "a standard source of ultra violet light composed of a certain lamp which is so kept that it is unlikely to change the candle power, be compared with the action of the lamp one wants to measure on one and the same culture of germs." He used a culture of paramecium. A drop of such a cuture is exposed at a definite distance from the laboratory standard quartz lamp and another drop at the same distance from the lamp to be tested. By recording the time necessary for killing, the relative value of the two lamps can be obtained. However, it has not been determined that bactericidal activity and chemical activity of the rays are proportional. This relation could probably be determined easily, and if found, might lead to more accurate quantitative determinations by means of chemical effects. Such quantitometric determinations would be much simpler in technic than the biological method.

Mercury vapor arc lamp. The richest known artificial source of ultra violet rays is the mercury vapor arc inclosed in a quartz tube. The principal advantage of this arc lamp over other lamps is the fact that no disintegration of the electrodes occurs. When the lamp is lighted, the electrodes are constantly being replaced by the condensed mercury vapor running back to the electrode container. The light is made by tilting the lamp so that the mercury forms a complete bridge from one electrode to the other, and then on releasing the tube the mercury runs back and draws out the arc. The lamp cannot run on alternating current because as soon as the value of the current sinks to zero the arc goes out and cannot restart except by re-striking. The life of such a lamp is stated to be from 1500 to 7000 hours. The production of ultra violet rays increases with the temperature of the lamp. Consequently, when first lighted the yield is small and a maximum is not reached for five or ten minutes.

Use in water purification. The first practical results in purifying drinking water with this light were obtained by Courmont and Nogier (6). They suspended their lamp in a metallic tank and with both naturally and artificially polluted water, obtained complete sterility in one to two minutes. The following year, Henri, Helbronner and von Recklinghausen (7) made experiments with water flowing at

125 cubic meters, or 33,000 gallons, per hour. Water polluted artificially with a rich emulsion of B. coli was passed through the fields of four lamps arranged in series. The samples taken beyond the second lamp were sterile. Bacterial spores, also, were found to be killed but showed considerably more resistance than vegetative forms.

At the present time this method is receiving considerable appli-

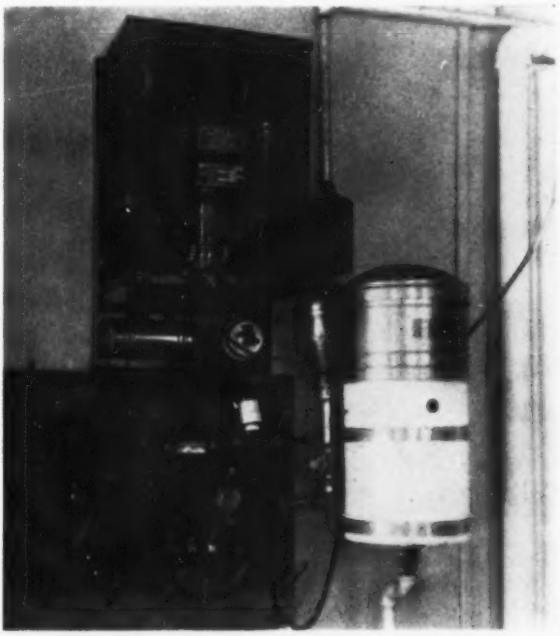


FIG. 1. ULTRA VIOLET RAY STERILIZING APPARATUS OF THE GRAVITY TYPE

Equipped with automatic tilted lamp and solenoid operated water valve so arranged that no water can pass the apparatus unless the current is on and the lamp at its maximum intensity.

cation in furnishing treated water for drinking purposes, in bottling works, in the sanitation of swimming pools, and for the use of armies in the field. In France the method is said to be successfully applied to the treatment of municipal water supplies.

Owing to the fact that ultra violet rays are readily absorbed by many organic substances and that their penetration into the water is hindered by the presence of suspended matter, it is necessary that

the water be perfectly clear, prior to treatment. For the purpose of removing turbidity, a rapid sand filter is usually employed.

Description of apparatus tested, with results. A gravity and a pressure type of apparatus were tested, Figs. 1 and 2. In the gravity type the lamp is suspended in a baffled treatment chamber a few inches above the surface of the water, which in a thin film is exposed twice during its passage by the lamp. This type is operated at 110 volts, and has a rated capacity of 120 gallons per hour.

The pressure type consists of a cast iron cylindrical shell with a quartz tube through its axis and containing an arrangement of

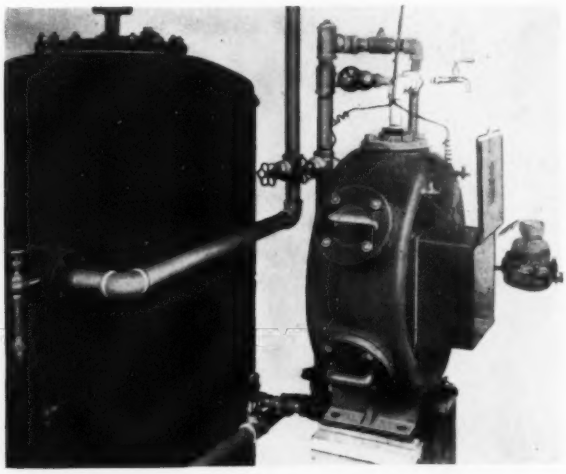


FIG. 2. LARGE PRESSURE TYPE ULTRA VIOLET RAY STERILIZING APPARATUS

Showing rapid sand filter, sampling cocks and water meter for measuring the rate of flow.

baffle plates for the same purpose as those in the gravity type. The lamp is inserted within the axial quartz tube. A 220-volt lamp is required to produce a sufficient amount of ultra violet rays for treating 750 gallons of water per hour, the rated capacity.

Usually a fluctuation in voltage of 10 per cent or over is sufficient to extinguish the light. To prevent the passage of untreated water, each apparatus is provided with an automatic electrical arrangement controlling a solenoid valve which will prevent the flow of water until the temperature of the lamp has risen sufficiently to produce

an abundance of ultra violet rays and which will cut it off when for any reason the light goes out.

Each apparatus was operated in connection with a rapid sand pressure filter which clarified the water just before it entered the light chamber. It was at this point that all the raw water control samples were taken. In order to artificially contaminate the water, the alum shunt-feed box on the filter was kept filled with rich broth cultures of B. coli and other bacteria. By this means enormous quantities of all kinds of organisms could be fed to the filter, which allowed nearly all to pass through to the ultra violet ray apparatus after the sand bed had once become thoroughly saturated with them. It can be seen from the accompanying tables that the raw water contained many times more B. coli than one ever finds in the worst drinking water.

The vast majority of all samples were collected while the apparatus was being operated in excess of its rated capacity.

The following table includes a total of 142 samples taken on 17 different days during a period of two months, with a small gravity type apparatus having a capacity of 120 gallons per hour.

NUMBER AND KIND OF SAMPLE	AVERAGE TOTAL COUNT PER CC. ON AGAR AT 37° C.	AVERAGE NUMBER OF B. COLI
33 raw water	29,400	3.256 in 1 cc.
109 treated water	12	1 B. coli in 15 cc.

86 of 109 treated samples gave 0 B. coli in 50 cc.
10 of 109 treated samples gave 1 B. coli in 50 cc.
2 of 109 treated samples gave 2 B. coli in 50 cc.
2 of 109 treated samples gave 3 B. coli in 50 cc.
1 of 109 treated samples gave 4 B. coli in 50 cc.
1 of 109 treated samples gave 5 B. coli in 50 cc.
7 of 109 treated samples gave 50 B. coli in 50 cc.

The last seven samples showing 50 B. coli in 50 cc. were taken after the machine had been idle for one month and the raw water controls at the same time had at least 100,000 B. coli per cubic centimeter and a total count of 150,000. If these seven samples are eliminated in figuring the average number of B. coli in the treated water, there would be only one B. coli in 175 cc. instead of one in 15 cc. as shown in the table.

One day's test on small gravity-type apparatus

LOCATION	HOUR	BACTERIA ON AGAR AT 37° C.			B. COLI IN LACTOSE BROTH												RATE GALLONS PER HOUR
		Plate 1	Plate 2	Average	0.0001 cc.		0.001 cc.		0.01 cc.		0.1 cc.		1 cc.		10 cc.		
					G	E	G	E	G	E	G	E	G	E	G	E	
F	10:20	48,000	53,000	50,500	+	+	+	+	+	+	+	+	+	+	+	+	105
S	10:20	4	0	2							0		0		0		105
S	10:25	2	2	2							0		0		0		105
S	10:30	3	3	3							0		0		0		105
S	10:35	4	2	3							0		0		0		105
S	10:40	1	3	2							0		0		0		105
S	10:45	7	9	8							0		0		0		135
S	10:50	6	8	7							0		0		0		135
S	10:55	7	10	8.5							0		0		0		135
S	11:00	6	13	9.5							0		0		0		135
F	11:00	51,000	50,000	50,500	+	0	+	+	+	+	+	+	+	+	+		135

F = filter sample; G = gas on lactose broth; S = sterilized sample; E = confirmation on endo media.

Apparatus ran fifteen minutes before first sample was taken.

The following table includes a total of 330 samples taken on 29 different days during a period of two months, with a pressure-type apparatus having a capacity of 750 gallons per hour.

NUMBER AND KIND OF SAMPLE	AVERAGE TOTAL COUNT PER CC. ON AGAR AT 37° C.	AVERAGE NUMBER OF B. COLI
48 raw water	3,900	2,630 in 1 cc.
282 treated water	7	1 B. coli in 103 cc.

211 of 282 treated samples gave 0 B. coli in 50 cc.
31 of 282 treated samples gave 1 B. coli in 50 cc.
21 of 282 treated samples gave 2 B. coli in 50 cc.
14 of 282 treated samples gave 3 B. coli in 50 cc.
3 of 282 treated samples gave 4 B. coli in 50 cc.
2 of 282 treated samples gave 5 B. coli in 50 cc.

One day's test on large pressure-type apparatus

LOCATION	HOUR	BACTERIA ON AGAR AT 37° C.			B. COLI IN LACTOSE BROTH												RATE GALLONS PER HOUR
		Plate 1	Plate 2	Average	0.0001 cc.		0.001 cc.		0.01 cc.		0.1 cc.		1 cc.		10 cc.		
					G	E	G	E	G	E	G	E	E	G	G	E	
F	10:10	4800	4500	4650	0		+	+	+	+	+	+	+	+	+	+	1270
S	10:10	0	0	0							0		0		0		1270
S	10:15	0	2	1							0		0		0		1270
S	10:20	1	0	½							0		0		0		1270
S	10:25	1	0	½							0		0		0		1270
S	10:30	3	1	2							0		0		+	0	1270
S	10:35	0	1	½							0		0		0		1270
S	10:40	2	0	1							0		0		0		1270
S	10:45	3	0	1½							0		0		0		1270
S	10:50	0	0	0							0		0		0		1270
S	10:55	1	2	1½							0		0		0		1270
S	11:00	1	2	1½							0		0		0		1270
S	11:05	0	0	0							0		0		0		1270
S	11:10	1	0	½							0		0		0		1270
F	11:10	3200	3700	3450	0		0		0		+	+	+	+	+	+	1270

F = filter sample; S = sterilized sample; G = gas on lactose broth; E = confirmation on endo.

The apparatus ran fifteen minutes before the first sample was taken.

The bacteriological tests were performed, with slight exception, in accordance with the Standard Methods of Water Analysis of the American Public Health Association. The fermentation of lactose broth and the characteristic appearance on Endo were considered sufficient identification of B. coli.

A total of 391 treated samples were tested. Of these 53, or 13.5 per cent, did not come within the Treasury Department standard of permissible impurity, which requires that not more than one of five 10-cc. portions of each sample shall show the presence of organisms of the B. coli group. This appears to be a poor showing. However, it cannot be so considered when a comparison is made with the extremely polluted raw water control samples collected at the same time.

In the gravity type of apparatus 86 of 109 treated samples, or 78.8 per cent, showed no organisms of the B. coli group in 50 cc., while the untreated water contained an average of 3256 B. coli per cubic centimeter.

In the pressure type 211 of 282 treated samples, or 74.8 per cent, gave no B. coli in 50 cc. while the untreated water was yielding 2630 B. coli in 1 cc. The great reduction is the more remarkable when one considers that the actual time of exposure of the water to the rays is approximately sixty-four seconds in the large, and thirty seconds in the small apparatus, when they are running at their normal rate.

The fact that spore bearing organisms are somewhat more resistant to these rays than vegetative forms, has been shown by von Recklinghausen. In the above outlined tests the lactose broth tubes would frequently show fermentation at the end of forty-eight hours. In such cases transfers invariably proved negative on Endo for B. coli. The organism that caused this result proved to be a strict anaerobe and was later identified as belonging to the B. Welchi group. It is believed that the appearance of such organisms in water is not of sanitary significance (14).

Conclusions. The use of ultra violet rays in the purification of drinking water is relatively a recent procedure, and it is believed that the method is capable of further development, with a corresponding increase in efficiency. The method is especially recommended for treating water in circulating systems, in which the water may be exposed many times to the ultra violet rays. Such systems are now used for disinfecting water in swimming pools, hotels, industrial, and other institutions. The chief advantages of this method of treatment over chemical methods lies in the fact that objectionable overdosage is impossible. Again, from the standpoint of potability, the water is absolutely unchanged, and hence, in this respect, is superior even to boiled water. Recently, ultra violet rays have been recommended for sterilizing the drinking water on vessels traversing the Great Lakes (15). The fresh water of the lakes is very clear, and the turbidity that does occasionally occur consists mostly of heavy suspended matter and is easily removed by filtration.

The present forms of apparatus need considerable attention when in operation. Care should be taken to prevent an accumulation of grease or dirt of any kind on the quartz tube. Experience has shown that a very small amount of grease will obstruct the light sufficiently to allow many organisms to pass. Furthermore, the light chamber should be emptied if the machine remains idle for any length of time. This will prevent the growth of algae and other

organisms which will obstruct the light when the machine is again operated. The latest types of apparatus are now arranged so that the quartz tube can easily be removed for inspection and cleaning. There is no way to determine when the lamp fails to emit a sufficient amount of ultra violet rays to give a maximum bactericidal effect, except by collecting samples and testing the treated water frequently. Theoretically, as long as the lamp remains intact and the vacuum holds, no reduction in the amount of ultra violet emanation occurs. It is stated by the manufacturers that a lamp will wear out before its efficiency noticeably decreases. The proper construction of the lamp, however, is of great importance and considerable variations have been noted between individual lamps.

In the above tests, since the gravity type gave a higher efficiency than the pressure type, and inasmuch as any lamp operating at a higher voltage is known to emit a larger amount of ultra violet radiation, the use of lamps at a higher voltage, combined with the gravity system, suggests itself as a reasonable line of development.

In regard to cost, the method is not expensive when treating relatively small quantities of water. For municipal supplies, using several million gallons per day, the expense is considerable, and probably prohibitive. The larger passenger vessels on the Great Lakes, which need a maximum of 25,000 to 30,000 gallons per day, have found the use of ultra violet to be the most economical method which meets the Government requirements for drinking water supplied to passengers in interstate traffic.

Up to the present time this method of treating water has not been applied very extensively; nor has it been developed to the extent that the known evidence in its favor would seem to indicate.

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SERVICE PIPES[1]

By F. N. Speller

Nearly every kind of pipe material has been used at some place or other for water service, cast iron, cast iron lined with enamel or glass, lead, zinc, galvanized wrought iron and steel, and wrought pipe with lead or cement lining. The report of the Committee on service pipe of the New England Water Works Association, September, 1916, contains some practical data on some of these varieties of pipe which are now in use, with particular reference to the value of certain protective coatings. In this brief paper I have thought it best to confine my remarks to certain developments in the manufacture and use of galvanized pipe for water service lines, recognizing the fact that there are other conditions where some more effective and more expensive protective coating, such as lead or cement lining, is required.

In discussing the deterioration of wrought pipe it will be convenient to consider (1) outside corrosion from soil, (2) internal corrosion from water. In each case it is important to know the relative durability of the unprotected metal by itself.

Wrought iron was the only form of iron which could be readily welded until about twenty-five years ago, when soft steel was developed to the point where it could be made into pipe. Since then this steel has been greatly improved and at present constitutes about 90 per cent of the wroughtpipe demand. It is important for the engineer and water works superintendent to form an accurate opinion on the relative durability and service ability of these two materials, which make up the cheapest and strongest pipe available at present. There has been so much discussion of an apparently contradictory character on this subject that one who has not given the complicated subject of corrosion some thought may easily be led to an erroneous conclusion. If opinions and hearsay evidence are eliminated and only comparative service under like conditions be admitted, it soon becomes clear that there is, in fact, very little dif-

[1] Read before the New York Section on February 21, 1917.

ference between wrought iron and steel as a class under most conditions, and the difference in opinion which has so long prevailed in the minds of honest observers may be satisfactorily explained. The extent of corrosion is found to depend on a number of external conditions, such as the quality of the water, temperature, and volume of flow, the presence of matter in suspension, contact of the pipe with other metals or the soil. Even the contact of dissimilar soils brought about in filling in the ditch will often be sufficient to set up electrolytic action and accelerate corrosion. This is a complicated matter and the relative influence of these factors cannot even be estimated, so that the only safe guide is to put both materials into service together and make a careful investigation of the results.

In considering outside corrosion, water works engineers may draw on the experience of gas companies. Some of these concerns have made a special study of this question during the past ten years and have put in both wrought iron and steel quite extensively for test purposes. A number of papers have been published in the *Proceedings of the American Gas Institute* giving detailed experience on wrought pipe under ground.* At this time, as a result of their experience and investigation, the majority of gas companies use steel pipe, some of which is protected with bituminous or combination bituminous and fabric coatings depending on conditions. Some public service companies have made a careful survey of soil conditions in their locality and have been able intelligently to apply coatings locally where conditions were found to warrant the additional expense.

A recent analysis of the pipe purchased by gas companies in 105 localities in New England showed the following, which is, I believe, representative of the present trend:

		per cent
Those using steel pipe exclusively,	75	71.42
Those using steel and wrought iron,	13	12.38
Those using wrought iron exclusively,	15	14.28
Unknown,	2	

Protection of the outside of pipe. Galvanizing has considerable value, but most gas service lines are coated with some form of bituminous coating. This over the galvanizing should be more effective. In unusually damp corrosive soil a layer or two of saturated fabric tightly wound on the pipe after the application of an impervious priming coat affords substantial protection.

* Proc. Am. Gas Inst.: Vol. 3—265, 274. Vol. 6, pt. 1—318, 348, 351, 357. Vol. 8, pt. 2—145, 154, 191, 212, 216, 223, 259. Vol. 9—1065.

In applying coal tar or asphalt to cold pipe the surface should first be dried and a priming coat applied, consisting of a thin mixture of coal tar or asphalt in benzol. When this has dried hot pitch may be applied, of a substantial thickness, which will adhere tightly to the cold metal. This should of course be done as near to the job as possible.

Internal corrosion. In the case of cold water services this depends primarily on the quality and flow of water. In some localities, where the water is slightly hard, no appreciable corrosion is found after twenty-five or thirty years, but in other localities, particularly when the water is soft and is saturated with oxygen and carbonic acid, the action is more rapid. However, in buildings, the experience with galvanized pipe (iron or steel) for cold water lines has been on the whole satisfactory, so that most architects now allow galvanized pipe on the cold water but require brass for the hot water line.

For this reason a test of the durability of pipe in cold water supply lines would take many years, and such tests should be made under a variety of water conditions. Many have taken advantage of the accelerated corrosion with heated water to compare various kinds of wrought iron and steel pipe in service, black and galvanized, so that there are now over one hundred and fifty such tests on record, all indicating that there is no fundamental difference in life between wrought iron and steel. There is no apparent reason why this experience does not apply to cold services using the same water.

Service tests of hot-water supply lines have recently been finished in the Pennsylvania Building, Philadelphia, in three of the New York City baths, and in the Irene Kaufmann Settlement, Pittsburgh, Pa. All were made under the supervision of independent investigators. A few typical pieces from the Irene Kaufmann Settlement are shown in Figures 1, 2 and 3, which are fairly representative of results obtained at the other places mentioned. The results of such service tests have never been questioned, even by wrought iron manufacturers, although the latter have offered the criticism that the contact of metals was unfavorable to the iron. This may or may not be the case, but even so, the influence of contact is to set up a very weak electromotive force which could extend only $\frac{1}{8}$ inch or so from the juncture. The extreme ends of the pipe always show more corro-

sion than parts farther in, due to this action or the erosion of eddy currents, and for this reason an inch or so of the ends are always discarded in measuring the corrosion. Another explanation as to

Average of 5 deepest pits, decimals of an inch,

0.121	0.128	0.121	0.111

Deepest pit, decimals of an inch,

0.126	0.131	0.127	0.117

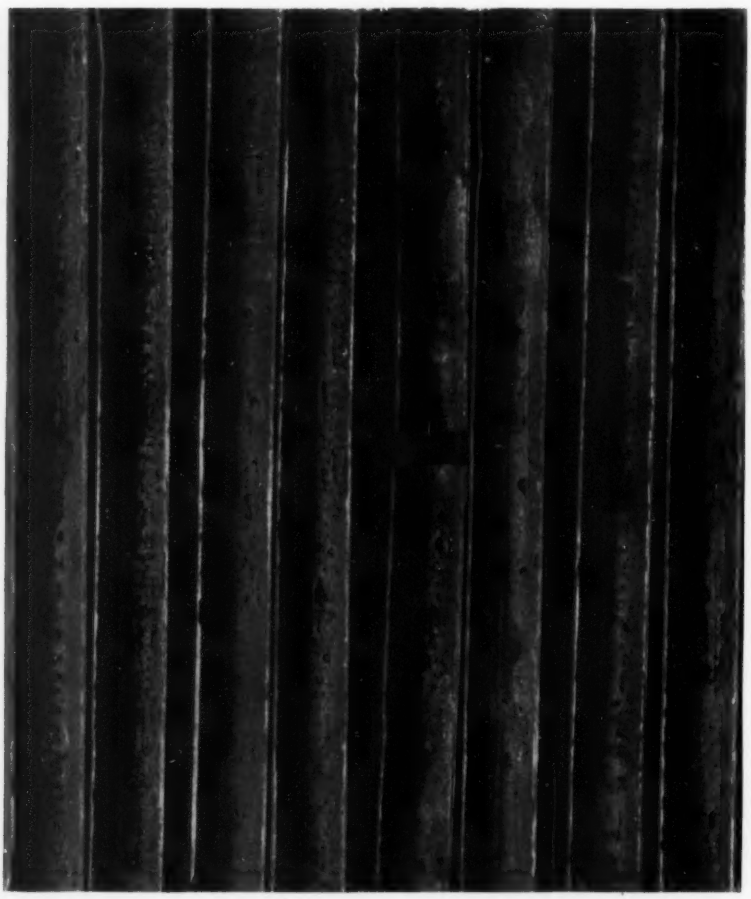


FIG. 1. CORROSION TESTS AT IRENE KAUFMANN SETTLEMENT
GENUINE WROUGHT IRON

why these tests have not borne out the expectations of some is that this particular iron was inferior. There are so many of such cases where the pipe was taken from commerical stocks that the average should be representative.

These tests of pipe carrying heated water also indicate that the galvanizing on wrought iron and steel pipe is practically the same in respect to its protective action on the metal. The zinc dis-

Average of 5 deepest pits, decimals of an inch,

0.123	0.123	0.107	0.110

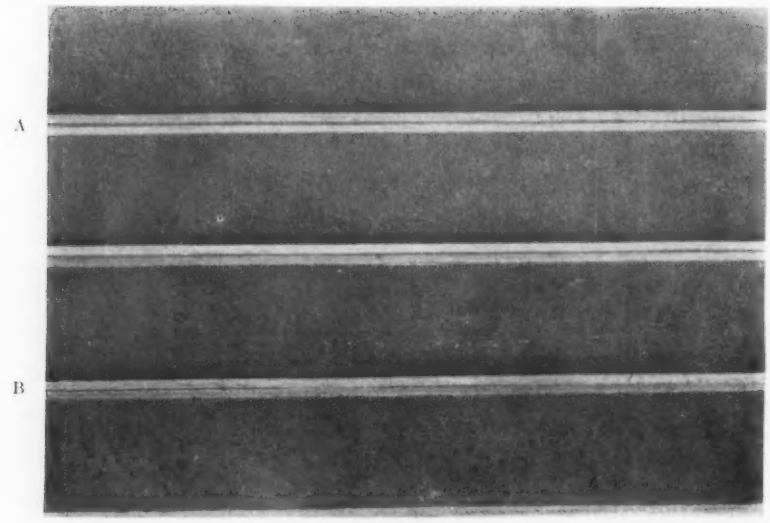
Deepest pit, decimals of an inch,

0.127	0.125	0.120	0.124

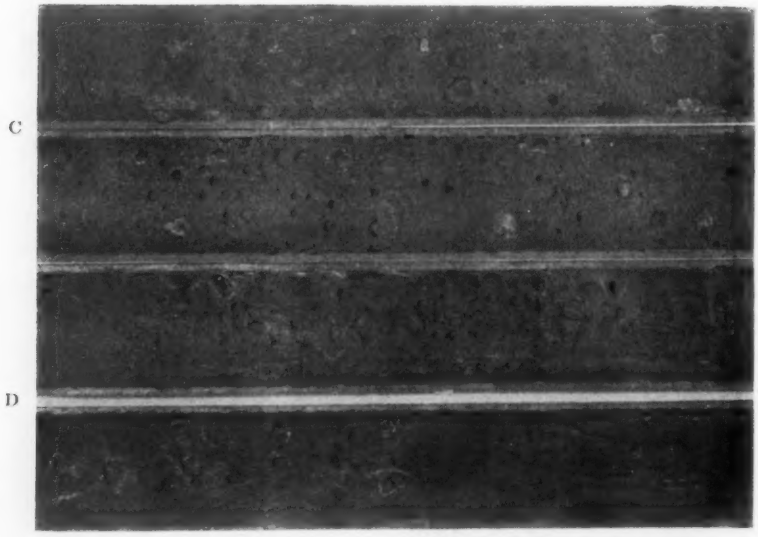
FIG. 2. CORROSION TESTS AT IRENE KAUFMANN SETTLEMENT
NATIONAL STEEL

solves rather rapidly leaving a zinc iron alloy which is alloyed with the iron.

As galvanized iron and steel are so generally used for services a word as to some practical features of galvanizing may be of in-



From deoxidizing system



From regular closed heating system

FIG. 3. COMPARISON OF PIPES OPERATING ON DEOXIDIZED AND NATURAL WATER

	A	B	C	D
Deepest pit, inch	0.019	0.047	0.127	0.126
Average of 3 deepest pits, inch	0.016	0.042	0.123	0.121

Note: Pieces A and C from same length of Steel pipe
Pieces B and D from same length of Wrought Iron pipe

terest. The most approved method as generally practiced by all pipe manufacturers in this country is known as the hot galvanizing process. The pipe is cleaned in acid, washed and neutralized in water, dipped in a fluxing bath and dried. It is then rolled into a bath of molten zinc which is maintained at a temperature of about 830°F., in which the pipe is allowed to remain until it acquires the temperature of the molten zinc. In the best practice no zinc is removed from the pipe, inside or outside, except what will naturally drain off when the pipe leaves the bath. Prime western spelter is generally used, averaging less than 1.5 % impurities. The weight of coating should average over 2 ounces per square foot of surface coated.

The uniformity of the coating depends on the cleaning of the surface in acid. This must not be carried too far or the metal will be "burned," so that the zinc will not alloy to the surface. Unfortunately the cinder formed in welding all pipe lies irregularly disposed on the interior surface and often carries siliceous matter picked up from the hearth of the furnace, which is like the enamel on cast iron in offering great resistance to the action of acid. The cleaning of the inside of buttweld pipe has been a very difficult problem, shared alike by all manufacturers of both wrought iron and steel pipe. A process for mechanically removing this scale from the hot pipe has recently been developed which promises to make possible considerable improvement in the zinc coating, and at the same time saves a large percentage of acid.

Zinc owes its protective power to its electro-positive character with respect to iron, i.e., a current is caused to flow from the zinc to the iron wherever the latter is exposed to water. No metallic coating so far made on a commercial scale is entirely free from pores or pin holes. Lead would be a better metal than zinc for this purpose except that any porosity in a lead coating leads to rapid destruction of the underlying iron, whereas zinc will protect iron an eighth of an inch or so away from the coating. Zinc forms an alloy with iron which possesses much less electro-positive power than pure zinc.

A Committee of the American Society of Testing Materials is now at work on specifications for galvanized coating. The unanimous opinion expressed with respect to the determination of the weight of zinc is that the lead acetate method for dissolving the coating is much preferable to the Preece or copper sulphate dip test.

SOME COSTS OF MAINTENANCE OF MOTOR-DRIVEN DEEP-WELL PUMPS[1]

By Melvin L. Enger

There seems to be very little published information concerning the cost of operation of deep-well pumps. The following notes, although not as complete as might be wished, may therefore be of value.

The water supply of the University of Illinois comes from five wells drilled about 140 feet deep in the glacial drift of this region. The water bearing stratum is about 15 feet thick and consists of sand containing much fine material. The water level in the wells when the pumps are not running is about 95 feet below the surface and when the pumps are running about 110 feet below the surface.

The water is pumped from the wells by means of motor-driven Luitwieler deep well pumps. The working barrels are $5\frac{3}{4}$ inches in diameter and the stroke is 15 inches. The pump on Well 2 is geared directly to its motor, the other pumps are belt-connected.

Table 1 gives information concerning the wells and pumps, and Table 2 the cost of labor and material to operate the pumps for sixteen months.

TABLE 1

Wells and pumps of the University of Illinois

	WELL NUMBER				
	1	2	3	4	5
Date drilled	1900	1907	1904	1905	1914
Depth, feet	138	140	142	140	140 (about)
Diameter, inches	8	8	8	12	12
Cost of well, casing and screen	$559	$546	$699	$1348	$891
Cost, motor and pump	$375	$813	$788	$758	$810
Capacity, g.p.m., 1914 tests	60	69	81	71	75
Speed, r.p.m.	31	33	28	30	30

[1] Read at a meeting of the Illinois Section at Urbana on March 13, 1917.

TABLE 2

Cost of labor and material for operating University wells during sixteen months

DATE	NO. 1		NO. 2		NO. 3		NO. 4		NO. 5	
	Labor	Material	Labor	Material	Labor	Material	Labor	Material	Labor	Material
1914										
October....	$11.11	$0.00	$14.15	$1.21	$9.11	$0.00	$15.03	$5.90	$3.43	$1.00
November.	2.60	0.31	3.10	0.00	7.95	1.25	5.15	0.00	35.87	13.18
1915										
January....	1.46	0.94	5.06	0.00	1.86	0.13	42.77	0.15	10.94	6.24
March.....	10.96	10.10	2.26	0.00	7.71	1.52	1.96	0.39	11.70	0.37
April.....	3.83	80.84	4.61	0.00	8.18	0.46	5.97	3.30	3.86	1.16
May.....	3.40	0.19	3.59	0.00	27.17	0.27	3.97	0.31	4.38	7.52
June.....	5.92	0.45	7.64	0.33	10.10	0.55	6.49	0.54	7.72	9.15
July.....	49.19	0.64	2.36	0.00	3.23	0.79	2.57	0.14	4.30	0.35
August.....	3.38	0.01	4.01	0.00	3.06	0.00	3.02	0.00	14.42	0.01
October....	6.11	1.64	2.27	0.36	3.80	0.94	6.74	1.09	18.88	0.79
November.	6.75	0.07	0.99	0.20	9.73	1.55	0.66	0.93	4.08	16.30
1916										
January....	5.44	0.07	1.91	0.00	2.44	0.35	1.96	0.15	3.65	0.15
February..	4.62	0.14	0.95	0.15	1.96	0.32	2.95	0.58	17.70	12.15
March.....	2.15	0.00	0.80	0.01	30.69	19.18	1.93	0.00	2.57	18.14
April.....	9.13	0.91	1.47	0.00	41.59	14.70	8.82	6.32	1.86	0.00
May.....	16.02	3.95	0.99	0.00	16.15	40.85	3.06	23.12	1.90	1.57
Total....	$172.07	$100.26	$56.51	$2.26	$202.73	$82.86	$113.05	$42.93	$147.26	$88.08

During the period included in Table 2 the wells were in operation the following percentages of the total time: No. 1, 44; No. 2, 34; No. 3, 70; No. 4, 86 and No. 5, 89. Taking the average, the wells were in operation 65 per cent of the total time, or 15.6 hours per day.

The pumps had been in use for periods ranging from one to eleven years, hence the results should represent the average performance of the pumps during their useful life fairly well. The principal cause of trouble is the fine sand getting into the well and into the pump. Rods, couplings, working barrel and packing are worn quite rapidly. The conditions under which the pumps work are decidedly poor, which accounts for the high cost of maintenance.

The total cost of maintenance for the five pumps during the sixteen months was $1008. This is equivalent to a cost of $151 per year for one pump operating 15.6 hours per day. Put in another way, it is equivalent to a maintenance charge of 0.82 cent per 1000 gallons pumped.

FLOW MEASUREMENT IN A WATER COLLECTING GALLERY BY THE CHEMICAL METHOD[1]

By D. H. Maxwell

Measurements recently made of flow in the infiltration galleries of the Des Moines Water Company are of interest as presenting a somewhat novel application of the chemical method, and showing the adaptability of this method to a situation where satisfactory discharge measurements by the velocity area method are not possible. The theory of the chemical or volumetric method is given in a paper by B. F. Groat, published in the 1914 *Proceedings of the Engineers' Society of Western Pennsylvania*. This paper is in part a review of methods developed by European enginieers. The same author last year presented a very exhaustive paper before the American Society of Civil Engineers, on the theory and practical application of the chemical method to precise testing of hydro-electric generators.

The chemical method of measuring discharge has been developed principally as applied to stream flow, particularly of mountain torrents, where current meter measurements are unsatisfactory, and for turbine testing, where its superior precision may make it preferable to other available methods. The extreme refinement used to secure high precision in turbine testing is apt to obscure the usefulness of this method in many cases where great precision is not required, and where it can be readily and cheaply used to advantage. It has a distinct field, for instance, in measuring flow in submerged conduits inconvenient of access, or in which the velocity is so moderate as to be practically outside the range of the current meter, as in the case of the Des Moines infiltration galleries.

Description of galleries. The total length of gallery completed and in use at the time of the test was 8480 feet. Manholes are provided about 600 feet apart. These galleries are built at a depth of from 20 to 25 feet in the very extensive and uniform coarse sand

[1] Read at a meeting of the Illinois Section at Urbana on March 14, 1917.

deposits of the Raccoon River valley, and are submerged 5 to 10 feet at low water. The newer part, 5160 feet in length, is built of concrete rings and has a cross-section of 12.6 square feet. The older galleries are rectangular and vary from 15 to 20 square feet in cross-section. When yielding at the rate of the present maximum pumpage, 14,000,000 gallons per day, the average velocity at the lower end of the main gallery is only 0.9 foot per second, the velocity decreasing progressively to practically zero at the far end.

The location of the two main gallery lines with reference to the river, the principal source of supply, is shown in Figure 1. The variation in distance from gallery to river, and the long continued use of the older galleries near or underneath the channel, with consequent clogging of the overlying sand, made it reasonable to suppose that the yield was not constant throughout the gallery length.

Purpose of test. In planning for the future extension of this supply, it was important to know the amount of variation in yield between different parts of the gallery from these causes. Measurements by current meter, made two years ago, were not entirely satisfactory because of the very low velocities in the gallery, and the difficulty of determining the average velocity of the cross-section. When the subject again came up for study last year, it was decided to measure the flow by the chemical method, as better adapted to the conditions.

Method of measuring flow. The test consisted in feeding a solution containing a known amount of salt (sodium chloride) at constant rate into one branch of the galleries near its upper end, and titrating for chlorine in samples of the gallery water collected at all manholes between the dosing point and the pumping station, including a sample at the pumps, just before the water entered the Venturi meter. The chlorine determinations were expressed in parts per million. These determinations, corrected for normal chlorine, indicated the dilution at any given point, of the salt fed into the gallery, and gave a measure of the flow at the corresponding point.

The equation for flow at any point is:

$$Q = \frac{C - c_2}{c_2 - c_1} q$$

in which Q is the gallery flow, q is the rate of dosing, C the concentration of the dosing solution, c_1 the normal concentration of chlor-

ine in the gallery water, and c_2 the concentration of chlorine in the gallery water at the sampling point, after dosing.

The volume of the dose was so small in comparison with the gallery flow, that it was ignored except to keep it constant. In other

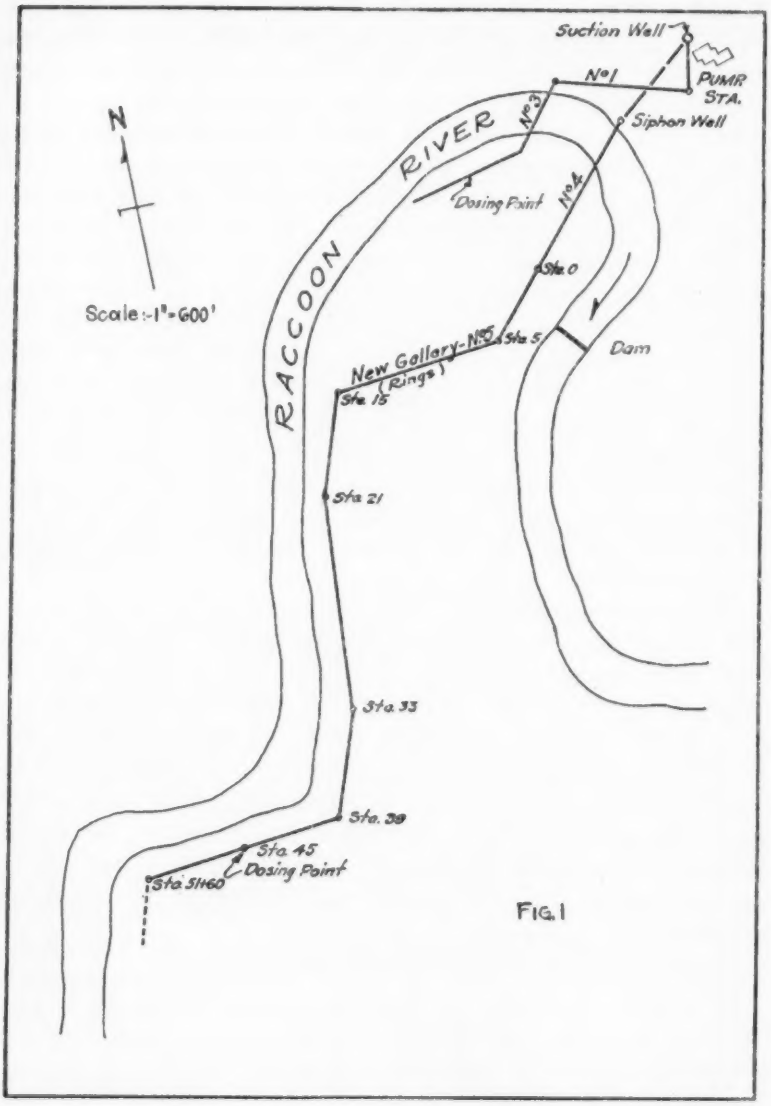


FIG. 1. LOCATION OF DES MOINES INFILTRATION GALLERIES

words, the salt used in making up the solution was carefully measured and was always added to the same quantity of water, but the exact amount of water in the solution was not determined.

Having determined the rate of flow at each manhole, the difference between the flows at successive manholes gives the total infiltration rate of the gallery intervening, and this divided by the length gives the average yield per unit length. This enables us to compare in rate of yield the several lengths of the gallery as divided by the manholes.

The ideal condition for this test, of perfectly uniform draft on the galleries, could not be secured. Des Moines has a direct pressure system and the pumpage fluctuates more or less continually. It was not practicable to overcome this difficulty by taking simultaneous samples at the several sampling points, so the test was continued during the greater part of the day, and the results used over selected periods of comparatively uniform pumpage, as shown by the Venturi meter chart. In this way the results are based upon observations made under conditions approaching as nearly as possible the ideal of steady uniformly accelerated flow, that would pertain under constant draft. It is apparent that any departure from this condition would, in the absence of simultaneous samples, give a false indication of the amount of infiltration between sampling points.

A check on the total yield of the galleries computed by this method was furnished by the Venturi meter. In fact, having the Venturi meter record of the total flow, it would not have been necessary for the purpose of this test to know the rate at which salt was added, although it was desirable to have this and use the meter as a check. The chlorine determinations might have been used merely to measure relative flow, and then compared with the chlorine determination at the Venturi meter and the flow as measured by the meter to compute the actual flow at any sampling point.

Details of test. Commercial sodium chloride was used for this work. The solution was prepared by dissolving 57 pounds of salt in about 35 gallons of water, as needed in each of two barrels from which the solution was drawn alternately. Each barrel was emptied in about thirty minutes. The barrels were piped into an orifice box, the water level in which operated a float valve controlling the barrel discharge. A $\frac{3}{16}$ inch orifice was provided under about 4 feet head, the idea in using a small orifice and large head being to

minimize the effect on the discharge of the slight fluctuations in head that were unavoidable with the float valve used. A 1-inch drop pipe with funnel at the upper end caught the solution below the orifice and delivered it to the center line of the gallery.

The dosing apparatus was not entirely satisfactory. Its principal defect was the imperfect arrangement for maintaining constant head on the orifice, and absolute uniformity of feeding could not be secured with it. Some trouble was also experienced with small splinters in what appeared to be perfectly clean salt becoming lodged across the orifice. This was overcome by straining the solution through cheese cloth as it entered the orifice box. This trouble with the feeding apparatus was the principal source of error in the work.

The quantity of salt used and the size of the orifice were predetermined to approximate the desired chlorine determinations in the final sample at the pumping station.

The dose was applied in the case of the main gallery test at a point 600 feet from the upper end, with satisfactory results. The average velocity at this point during the test was about 0.113 foot per second. In testing the shorter gallery, the dose was applied only 350 feet from the dead end, and the results seemed to indicate that the velocity at this point (about 0.03 foot per second) was too low for very satisfactory results.

Sampling. Samples were collected at intervals of about one hour throughout the test, in 200-cc. bottles which were lowered into the manholes and opened as nearly as possible at the gallery center line. This precaution was necessary as at all the manholes there was considerable water standing in the shaft above the gallery, and this water doubtless lagged behind the main gallery stream in chlorine content. The care required in taking the sample and the distance apart of the half dozen or more sampling points, made it impracticable to take simultaneous samples, which would have made it possible to have conducted the test in less time.

Chlorine determinations. The samples were titrated with silver nitrate.

In the main gallery test it was observed that during constant pumpage the chlorine determinations at each sampling point continued practically constant (Fig. 2), giving a series of approximately parallel lines for the several sampling points. These values were averaged for each point.

In testing the shorter gallery, the dose, as already mentioned, was applied at a point where the velocity was evidently too low for the best results. Diffusion of the salt interfered with the appearance of constant chlorine concentration at the sampling points,

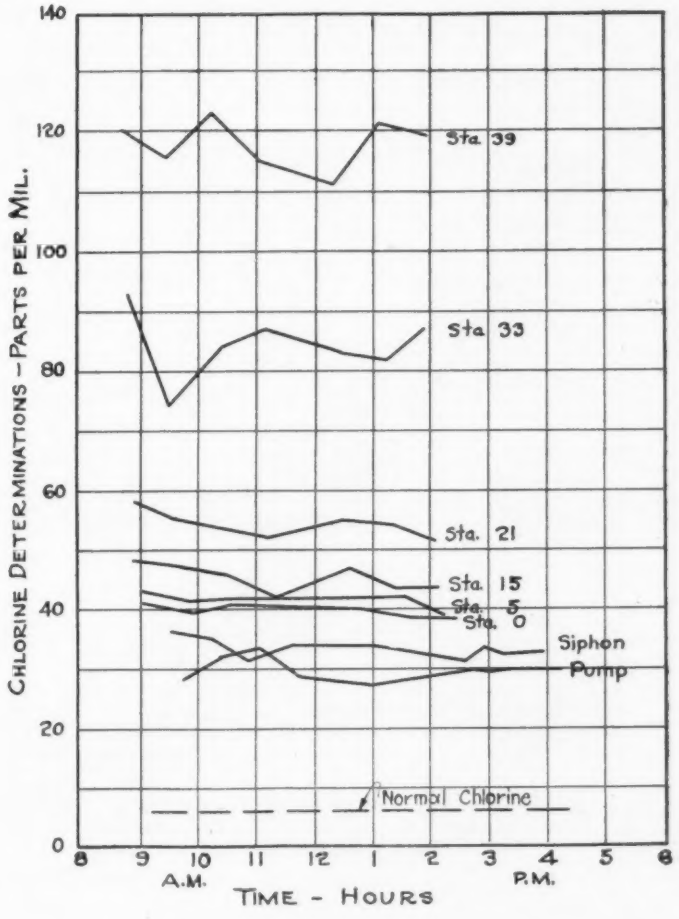


FIG. 2. CHLORINE DETERMINATIONS IN MAIN GALLERY

even under constant draft on the gallery, and somewhat obscured the results.

Results of test. The results of the tests are shown on Fig. 3. The infiltration rates under constant draft on the galleries varied

from 0.20 to 1.25 gallons per minute per foot length of gallery. By comparison with the plan showing gallery location (Fig. 1) it will be seen that the low rates were found in those sections of the gallery not adjacent to the river.

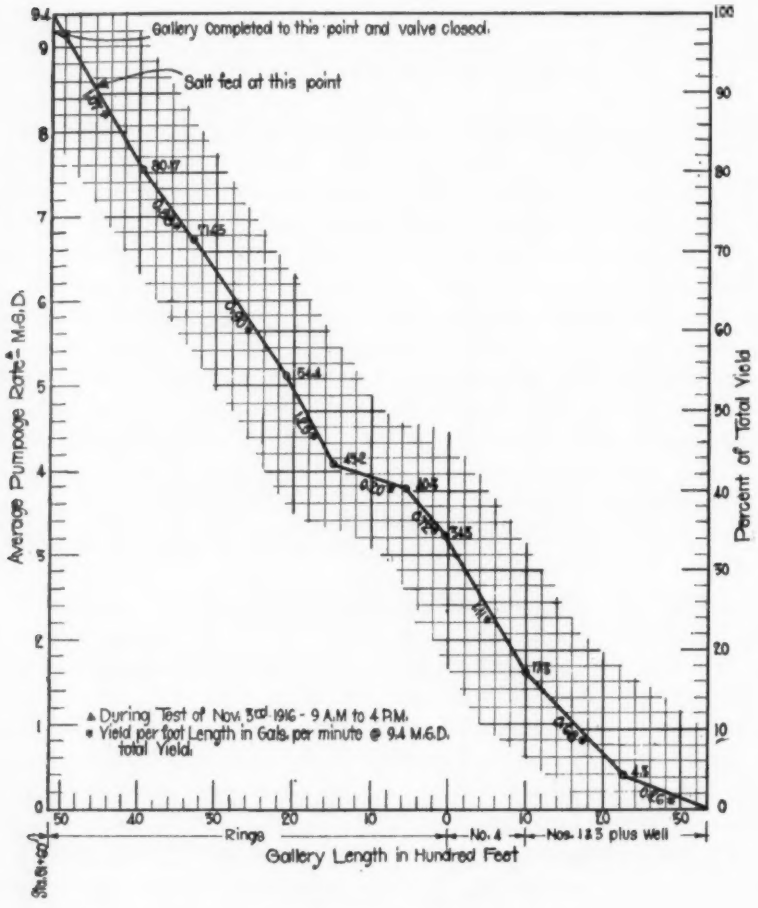

Gallery Completed to this point and valve closed.
Salt fed at this point
Average Pumpage Rate in M.G.D.
Percent of Total Yield
▲ During Test of Nov. 3rd 1916 - 9 A.M. to 4 P.M.
• Yield per foot Length in Gals. per minute @ 9.4 M.G.D.
total Yield.
Rings
No. 4
Nos. 1 & 3 plus well
Gallery Length in Hundred Feet

FIG. 3. YIELD OF DES MOINES INFILTRATION GALLERIES

The test was carried out in coöperation with Robert N. Kinnaird, superintendent, and E. T. Kirkpatrick, chemist, of the Des Moines Water Company, and was greatly facilitated by the admirable laboratory equipment of the company.

DISCUSSION

JOHN W. ALVORD: It may be of interest to outline something of the circumstance which required this investigation described by the author. The city of Des Moines obtains its water supply from a collecting gallery running in a remarkable formation of coarse sand and gravel beneath the Raccoon River valley. Ordinarily the filter gallery is looked upon by most sanitary engineers with suspicion, and in many cases it has undoubtedly failed to permanently produce water in sufficient quantities for the needs of the community for which it was projected. The Des Moines gallery, however, has proved to be an exception to this rule. This largely occurs because low floods on the Raccoon River clean off the sand beds of that river, and allow more or less permanent filtration to proceed.

Inasmuch as a very large monetary investment was under contemplation for these galleries, it became necessary to determine whether the present investment had met all the expectations formed for it. As it was impossible to measure the flow by any ordinary means, the interesting experiment related by the author was successfully tried. This has demonstrated that the gallery system as applied in this locality is permanent, valuable, and feasible.

THE QUALITY OF WATER AND CONFIRMATORY TESTS FOR B. COLI[1]

By Abel Wolman

The reliability of the presumptive test as an index of the presence of B. coli in water in the light of recent investigation, has been seriously questioned. The United States Hygienic Laboratory in its Potomac River report finds that the presumptive test varies in its importance as a measure of pollution, with the degree of pollution itself. Dr. Frost, in the Ohio River investigation,[2] concludes that the error in the presumptive test is greatest in the examination of treated waters. A. H. Creel[3] and Edward Bartow,[4] working on waters of similar character, that is, on drinking water from railroad trains, obtained results entirely at variance. The former, for instance, confirmed only 21 per cent of positive tests for gas formation, while the latter isolated B. coli in 83 per cent of the tubes showing gas formation. Graf and Nolte[5] conclude, on the other hand, that the bile test is a better index of the presence of B. coli the more polluted the water.

The author, having at hand analyses of various types of waters in the State of Maryland, made during periods in 1915 and 1916, thought it would be of interest to tabulate the results obtained in colon determinations and to attempt to draw possible conclusions regarding the efficacy of presumptive tests as an index of the presence of B. coli. In Table 1 the results for the raw waters and plant effluents of three filtration plants have been tabulated. The raw waters of all three are from surface streams, the pollution of the watersheds of each varying in intensity according to the rating given in the table. This rating has been based upon a knowledge of sanitary conditions resulting from a study of the three watersheds. It is of striking interest to note that the percentage of tubes con-

[1] Read before the Richmond Convention, May, 1917.

[2] *American Journal of Public Health*, Vol. VI, No. 6, June, 1916.

[3] *Hygienic Laboratory Bulletin No. 100*, pp. 43–57.

[4] *Journal of the American Water Works Association*, Vol. 2, No. 1.

[5] *American Journal of Public Health*, Vol. VI, No. 10, October, 1916.

firmed arrange themselves in the same grouping, A, B, C, as the sanitary survey of the streams, in a qualitative manner only, had already indicated. If we were to attempt to rate these raw waters by presumptive tests alone, it is at once obvious that the arrangement would be different and apparently not as accurate, since a study of the isolation tests indicates that the measure of the pollution of the streams varies in the same way as determined upon by the percentage of tubes confirmed. In the effluent samples from these same plants, the order of decreasing purity, as determined by presumptive tests, is C, A, B; by isolation tests, A, C, B, and by percentage of tubes confirmed, A, B, C. Assuming that isolation

TABLE 1

Raw water and effluents of three filtration plants

	RATING	NUMBER OF SAMPLES	PER CENT OF SAMPLES Positive Presumptive		PER CENT OF SAMPLES Positive Isolation		TOTAL NUMBER OF TUBES SHOWING GAS	PER CENT OF TUBES CONFIRMED
			10 cc.	1 cc.	10 cc.	1 cc.		
Raw water	A	56	96.7	89.5	87.7	69.8	152	82.3
	B	31	83.7	77.3	70.8	64.4	75	75.8
	C	168	100.0	77.9	72.5	51.2	383	67.9
Effluent	A*	106	50.9	30.2	40.5	18.9	95	70.3
	B*	58	12.0	1.7	6.9	1.7	9	66.6
	C†	169	60.9	7.1	10.7	4.1	266	23.7

*Untreated. †Treated.

tests for B. coli serve as a legitimate quantitative index of pollution, an apparent discrepancy in the scoring of the plant effluents would seem to appear. The explanation for such a discrepancy, however, would seem to the writer to be due to the insufficient number of tubes showing gas in plant B effluent, making its use of little statistical value.

Table 2 offers material for an interesting study of the interrelation of the quality of water and the percentage of presumptive tests confirmed. In this table the percentage column was first arranged in the order as given and the types of water, from which the results were obtained, were then noted opposite each value. The order in which the various characters of water fall is, in four of the

five instances, in agreement with what we should expect from a qualitative judgment of water unsupported by analyses. The assumptions in grading of waters 1, 2, 4, and 5 are borne out by the percentage of tubes confirmed. The position of water 3 (unfiltered, treated) on the same basis of scoring by percentage confirmed, is predetermined and its location is a curious one, establishing as it does the conclusion that an unfiltered treated water is somewhat less polluted, judged by this arbitrary method of scoring, than a filtered water which is not treated. Water 4, as noted in the table, is a mixture of treated filtered and unfiltered water. A qualitative estimate of this type of water would place it in a position between untreated filtered and treated filtered waters. The percentage of tubes confirmed establishes water 4 in such a position, and here again corroborates the accuracy of a method of scoring upon a

TABLE 2

Comparison of condition of water with percentage of tubes confirmed

RATING	CONDITION OF WATER	NUMBER OF TUBES	PER CENT CONFIRMED
1	Raw untreated	610	75.4
2	Filtered untreated	227	68.5
3	Unfiltered treated	37	44.7
4	Mixture of treated filtered and unfiltered	775	24.3
5	Treated filtered	266	23.7

"percentage of confirmatory tests" basis. It is of more than passing interest that water 3 is situated as it is, when we note that its score, 44.7, is almost the exact mathematical mean of the scores of filtered treated and filtered untreated. This would seem to indicate that in the particular waters under discussion the efficiency of filtration about equalled that of disinfection in the removal of the colon bacillus.

After having tested the qualitative estimates of waters by the above system of scoring, it was further elaborated by comparing the scores obtained in Table 2 with a grouping according to total bacterial content. Unfortunately a change in routine analytical determinations, within the periods used by the author, had occurred, which necessitated the use of both 37° and 20°C. counts. The arrangement of the different waters as made in Table 3 seems to be justified, in spite of the lack of adequate total counts of similar

temperature growth, particularly since a transition point was obtained by the fortunate existence of both 20° and 37°C. counts in the case of Water 3. In this table, too, the indications of pollution by the quantity of bacteria are borne out entirely by the percentage of tubes confirmed.

If the percentage of tubes confirmed is a real index of pollution, it would be expected to vary in the same way and with the same

TABLE 3

Relation between average counts and percentage of tubes confirmed

RATING	CONDITION OF WATER	AVERAGE COUNT		PER CENT CONFIRMED
		37°	20°	
1	Raw untreated	18,200		79.1
2	Filtered untreated	570		68.5
3	Mixture of treated unfiltered and treated filtered	45	534	24.3
4	Filtered treated		194	23.7

TABLE 4

Mechanical filter plant; comparison of atmospheric and raw-water conditions

DATE	TEMPERATURE	RAINFALL	PER CENT OF SAMPLES ISOLATED IN 1 CC.	PER CENT TUBES CONFIRMED
1915	°F.	*inches*		
October	57.9	2.86	62	61.3
November	45.2	1.48	34	42.3
December	33.5	3.36	67	77.0
1916				
January	38.0	1.36	27	52.5
February	32.6	2.79	38	71.7
March	36.0	3.26	45	56.2
April	51.5	2.69	73	78.4
May	65.2	4.65	69	97.8

degree of sensitiveness as those factors which predetermine it and are, de facto, indicative thereof. Of the factors producing abrupt changes in colon content, none is perhaps relatively more important than the rainfall. It can with justice be assumed that colon content in a general way varies simultaneously with the degree of rainfall. Here, then, we have another basis upon which to test our assumptions of variability of pollution with concomitant variability of confirmation tests for colon. In Table 4 there have been ar-

ranged, for a particular plant A, temperature, rainfall, colon content, and confirmation determinations for a period extending over eight months. With almost no exception, every increase in percentage of tubes confirmed occurs with an increase of total rainfall,

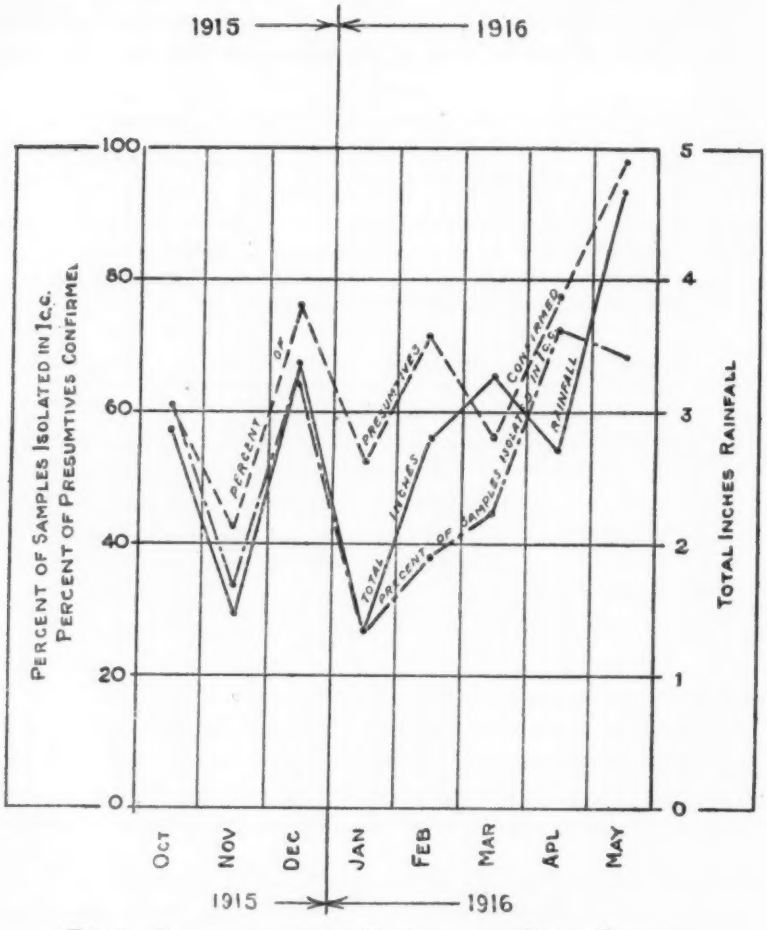


FIG. 1. RELATION BETWEEN RAINFALL AND COLON CONTENT

and vice versa. A failure of this agreement in April is explained when we observe that during this month the temperature had risen to a large degree, producing a condition of thawing and a consequent increase of colon content. The agreement of the colon con-

tent determinations by isolation in 1 cc. with the rainfall data is not as well defined throughout the period as is that of the percentage of tubes confirmed. In other words, upon this basis, the percentage of tubes confirmed shows a more sensitive effect of rainfall than do the actual colon contents. The results discussed here are shown graphically in the chart.

The conclusions which the above data seem to justify are, that:

1. A water showing the highest degree of actual pollution, as determined by the highest percentage of samples giving a positive isolation test for colon, in general gives the highest percentage of presumptive tests confirmed.

2. A water showing the lowest degree of actual pollution in general gives the lowest percentage of presumptive tests confirmed.

As a result of the above conclusions, a rough quantitative test for pollution, in a general study of a water, might with safety consist only of a determination of the presumptive tests confirmed, rather than of a detailed estimate of the percentage of samples showing isolation tests in varying dilutions. In such a procedure as outlined in the preceding pages, one might establish as bases for comparison a maximum and minimum percentage of tubes confirmed, using for this purpose waters grossly polluted, in the first case, and unquestionably pure in the second. With these maximum and minimum values as standards, the relative position of the water under consideration could be determined with ease by the use of the "percentage of tubes" scoring method.

The use of such a system of scoring as above outlined should, the author believes, bear more intensive study, although the method is of passing interest as an additional indicative factor of water pollution.

THE DETROIT WATER WORKS[1]

By Theodore Alfred Leisen

The growth of any public utility usually parallels the history of the municipality which it serves, and in conformity with this axiomatic fact the development of the Detroit water supply system has kept steady pace with the varying stages of growth of the city, until at the present time the effort to maintain this pace is evolving a tendency to break all records and, figuratively, to exceed all normally established speed laws.

Historical. The water works of Detroit are of comparatively recent origin, their history dating back only to 1825, when, by virtue of a grant of the common council, certain citizens were accorded the exclusive right to supply water to the 1,500 inhabitants which then constituted the city's population. Two small pumps were placed at the foot of Randolph Street, from which place wooden pump logs, of $4\frac{1}{2}$ inches internal diameter, were extended to Randolph and Jefferson Avenue, where a 9,600-gallon wooden tank was erected as a storage reservoir. The development of this embryo plant for several years was not remarkably rapid although some evidence of growth was manifest. In 1831 the concessionaires were incorporated as the Detroit Hydraulic Company, and an added impetus was imparted to the extension of the plant by the construction of a 120,000-gallon reservoir, the erection of a new pumping station, the installation of a 20-horse-power pumping engine, and a gradual but limited extension of tamarack water mains.

The control of the works remained in the hands of this company for five years, and during this time the venture proved a financial burden instead of a source of profit. The management was subjected to continual public criticism both on account of the quality of the water and the inadequacy of the supply. The culmination of this criticism was the appointment by the common council in 1836 of a special committee of investigation, whose report declared that the company had forfeited all of the rights and privileges

[1] Read before the Illinois Section at Urbana on March 13, 1917.

granted it under its charter, and recommended the purchase of the water works by the city. The recommendation was formally confirmed, and on June 4, 1836, the city acquired the plant for $20,500. The management of the newly acquired water works was vested in a committee of the common council, but municipal ownership did not eliminate the adverse criticism, and intermittent agitation on the part of the citizens resulted in the appointment in 1852 of a board of trustees, followed in 1853 by an act of the legislature creating the board of water commissioners, since which time the management of the water works has been under its complete charge.

Passing over the various periods of development and changes, and the abandonment through obsolescence of many of the earlier portions of the works, the actual incipience of the present plant may be said to date from 1873–74, when the property now known as Water Works Park was purchased, and construction commenced on several of the features designed to constitute the basis of the present works, including the sedimentation basin, the original pumping station; the first 24,000,000-gallon compound beam engine, and the first 42-inch cast iron main. Rapid increases in the equipment of engines and boilers, of intake pipes and tunnel, and of extensions of the main arteries to the distribution system came, with each succeeding year, as a natural sequence to the steady growth of the city, until today there exists a plant, which, if only the legitimate use of water were strictly observed, would suffice for a city of over a million inhabitants.

Statistical. The water works, as completed to date, are equipped in all of their ramifications to furnish an average daily supply of over 160,000,000 gallons, and a maximum of over 200,000,000 gallons per day, and distribute it to and beyond the extreme boundaries of the city as constituted a year ago.

The pumping equipment consists of three compound beam engines and seven vertical triple expansion engines, of capacities from 24,000,000 to 30,000,000 gallons each, and a combined pumpage capacity of 260,000,000 gallons daily. The boiler plant furnishing steam to the pumping engines consists of fifteen units of from 225 to 400 horse power each, or a total of 4400 boiler horse power, a capacity ample for the simultaneous operation of all the pumping engines with a safe marginal reserve.

The distribution system comprises 1040 miles of mains from 4 to 48 inches in diameter, of which about 100 miles are from 12 to

36 inches, and 50 miles are 42 and 48 inches in diameter. The service connections number 140,000, and represent a total length of approximately 800 miles, making a combined length of pipeage from the pumps to the consumer of 1840 miles. Embraced within the distribution system, exclusive of service connections, are 12,000 gate valves and 850 blow-off valves, and there are 7064 fire hydrants and 517 fire cisterns.

Operation. The city has been divided into low and high pressure districts. The low-pressure district embraces all territory below a contour line approximately 40 feet above the level of the Detroit River, and the high pressure district everything between the 40 and the 60-foot contours. The pressures maintained at the pumping stations average 47 pounds for the low service and 65 pounds for the high service, resulting in an average pressure of about 30 pounds at the ultimate points of distribution at normal stages.

The water supply is taken in at an intake crib in the Detroit River near the upper end of Belle Isle, 3200 feet south-eastwardly from Water Works Park, and flows thence through a 10-foot diameter brick tunnel to the shore shaft, whence it is distributed either into the settling basin, or through the by-pass conduits into the various conduits leading to the water galleries of the pumping stations.

At the shore shaft the water is treated with chlorine gas as a disinfectant, in proportions of from $1\frac{1}{2}$ to $2\frac{1}{2}$ pounds of liquid chlorine to a million gallons of water, or in the ratio of from 0.2 to 0.3 part of chlorine to 1,000,000 parts of water. The result of the disinfecting treatment has been the reduction of bacteria from an average of 187 per cubic centimeter in the river water to 14 in the treated effluent, and a reduction of 86 per cent in B. coli.

After the chlorine treatment the water flows normally through the so-called settling basin to the several conduits, and through four screen houses, with wire screens of $\frac{1}{2}$-inch mesh, and thence to the water galleries and suction pipes of the pumps. Thence the water is forced by the pumps through four 48-inch and five 42-inch mains into all of the branches of the distribution system.

The total quantity of water pumped during the past year, as indicated by Venturi meters on the force mains, was 46,689,000,000 gallons, equivalent to an average of 127,565,000 gallons per diem, with hourly rates of pumpage varying from a minimum equivalent of 72,000,000 to a maximum of over 200,000,000 gallons per diem.

The per capita consumption, based on a carefully estimated population of 781,133 for Detroit and the suburbs supplied, was 163 gallons, a quantity largely in excess of what is adequate for all legitimate purposes according to most reliable statistics. In an effort to curtail unnecessary waste all new services are metered and old connections are being metered as rapidly as possible. There are at present 60,000 metered services, or 43 per cent of the total, and as this percentage increases, the per capita consumption should be proportionately reduced, with a consequent saving both in cost of operation and in expense for new equipment.

Recent and future improvements. During the past two years the energies of the water department have been directed principally towards the extension of the distribution system, and special efforts made to extend the large supply mains with a view to affording an increased volume of water in sections of the city remote from the pumping station, and in localities where the consumption was greatest, with a resultant stabilization of the pressures. Since July, 1914, approximately 130 miles of water mains of all sizes have been laid, including over 16 miles of mains 42 and 48 inches in diameter, and the plans for the current year contemplate a further extension of over 13 miles of 42 and 48 inch mains in addition to about 80 miles of laterals 6 to 24 inches in diameter.

The result of the addition of these large arteries to the distribution system has been a decrease in the velocity through the pipes, with a corresponding lessening of the tendency of breakage in the large mains which previously may have been caused by water hammer aggravated in its action by the excessive momentum of abnormally high velocities. The construction of the large lines in the form of loops was, it is believed, a further factor in removing the tendency to sudden shock, by creating a more evenly balanced condition of flow.

The principal single extension planned for 1917 is a new distributing main 48 inches in diameter and 10 miles long, from the pumping station to a point where it will connect with other large mains and extend thence in gradually diminishing sizes. This will make 10 force mains radiating from the pumping station, five 42 inches and five 48 inches in diameter, whose combined capacity at safe normal velocities would be upwards of 300,000,000 gallons in twenty-four hours.

The new pumping station completed two years ago was designed

to house six 30,000,000-gallon triple-expansion pumping engines, and coincident with its construction, three 30,000,000-gallon pumps were installed. The old station contained seven engines having a combined capacity of 170,000,000 gallons, and the 90,000,000 gallons of the three new pumps brought the total pumpage capacity to 260,000,000 gallons.

In January, 1917, a contract was made for two new pumping engines of the same general type as those previously installed in the new station, but designed to pump 37,000,000 gallons each at normal speed, and 40,000,000 gallons at 22 revolutions per minute, under a guarantee of 190,000,000 foot-pounds duty per 1000 pounds of steam. The first pump is to be ready for service by January, 1918. The completion of these two engines will give an available pumpage capacity of 334,000,000 gallons, or with two units out of commission for repairs or other causes, a dependable capacity of 270,000,000 gallons daily.

Plans are now being prepared for a new boiler house to be erected at the rear of the new station, which ultimately will contain six 600 horse power boilers equipped with superheaters and automatic stokers, and all other modern appliances designed to assist in scientific and economical operation. The building will be in architectural consonance with the New Pumping Station, to which it will be attached by a connecting wing containing office rooms, shower baths, toilets and locker rooms.

Overhead coal bunkers holding ten days' supply of coal will enable the stokers to be fed with a minimum of manual labor, and steam ash ejectors will remove the ashes to a conveinently located bin, from which they can be loaded directly into cars or wagons. A brick stack 12 feet in internal diameter and 250 feet in height will provide ample draft for all the boilers when operating at over rating.

Filtration. The quality of the Detroit water supply is remarkably good for a surface supply obtained from a source contiguous to a thickly inhabited section, but it is not all that might be desired from a standard of purity, and in consequence the question of filtration is one which for some years has agitated the minds of those who were intelligently interested in devising means to improve the quality of the supply. The chlorination treatment, while highly desirable in the absence of a better method, should only be considered in the nature of a temporary expedient, and performs no function in reducing turbidity or removal of organic matter from the water.

The typhoid death rate, approximating 12 per 100,000 of population, as recorded for the past few years, cannot consistently be offered as the basis for a hysterical arraignment of the character of the drinking water; but the reduction of typhoid fever is only one of several reasons, sanitary and aesthetic, which can be advanced in favor of filtration. The fact that the water supply in its present condition is not entirely satisfactory, is subject to constant criticism, and is open to the possibility of occasional serious pollution, imperatively demands that it be improved. As filtration is admittedly recognized by all who are conversant with the subject as the most efficient method of treating a public water supply, and in conjunction with partial or limited sterilization can be relied upon to convert the river water into a perfectly clear, wholesome, pure article, every dictate of reason seems to call for filtration. Carried to its ultimate analyses, the problem resolves itself into these questions: Are the citizens of Detroit satisfied to accept an inferior article in the water furnished them, or will they demand the best that can be procured, and, will a cost of less than ½ cent per thousand gallons, the maximum cost of filtration, exclusive of interest and depreciation, deter them from exacting their demands?

Imbued with the conviction that the water supply should be purified, two propositions have been authorized which should prepare the way to a definite solution of the question of approximate cost both of construction and operation, before final steps are taken to install a filter plant for Detroit; the construction of a small experimental filter and the preparation of plans to show the proposed layout of a complete plant.

The experimental filter, which is practically complete, consists of a coagulating basin with a capacity of 18,000 gallons, and two small filter beds, each capable of filtering from 60,000 to 80,000 gallons per day. The principal object of this model is to demonstrate the maximum rate at which the water of the Detroit River can be filtered, the minimum quantity of coagulant necessary, and the requisite period of coagulation, to produce the desired results. With fairly complete records of results attained with waters of high turbidities and excessive bacterial content, it is reasonable to assume that equally satisfactory results may be secured with the comparatively clear and pure water of Detroit River, at a much higher rate of filtration, which will mean a considerable saving in the area of the filters, with a proportionate saving in cost of con-

struction, and the same principle may apply in the use of coagulants, with like results in operating costs.

The tentative plans for the filter plant provide for a low-lift pumping station; a coagulating basin with its necessary mixing chambers; a filter house having 80 filter beds; a chemical house for storing and mixing the coagulant, and a filtered water reservoir. The filtered water reservoir will occupy the space of the present settling basin, but the remainder of the plant will be located at the rear of the New Pumping Station. With the exception of the filtered water basin, all of the plant will be under one roof, but of varying elevations to conform to the requirements of the different units. The low-lift pumping station will be equipped with steam turbine driven centrifugal pumps capable of raising 400,000,000 gallons from the intake level to the coagulating basin. Steam for the turbines will be taken from the boilers in the new boiler house, and the raw water supply will be received through a 12 foot diameter conduit connected both to the 10-foot tunnel and the 6-foot conduit which parallel the easterly side of the settling basin. A screen house with motor-operated revolving screens will intercept floating ice and other coarse material in the water, before it reaches the suction well.

The coagulating basin and mixing chambers will occupy a space 400 feet wide, and 800 to 1000 feet long and have a capacity of 40,000,000 to 50,000,000 gallons, providing for a four-hour period of coagulation for 300,000,000 gallons daily. The basin will be of concrete, and provided with necessary baffles and division walls for adjusting the period of coagulation. The filter proper will consist of eighty beds of about 1200 square feet each, arranged in eight tiers of 10 beds with pipe galleries between tiers, the whole covering a space 300 by 400 feet. The full equipment of filter beds will not be required at the start, and probably 20 beds will be left for future development as the increasing consumption may demand. The chemical house will be placed between the coagulating basin and the filters, rising to a higher elevation for the storage and mixing space, and will be surmounted centrally by the wash-water tower.

The filtered water basin will be 350 feet wide, 800 feet long, and 20 feet deep, giving a capacity of 40,000,000 gallons. It will be all concrete construction, covered and parked over the top and laid out in tennis courts or ornamental shrubbery and lawn.

The plans in all their features contemplate a daily capacity of

over 300,000,000 gallons, with provision for meeting peak loads of 10 per cent. above the maximum normal conditions. It is difficult to predict what the future will require, but when the average daily consumption approaches 300,000,000 gallons, a second station located farther up the lake and equipped with large centrifugal units will probably be the solution.

A MUNICIPAL WATERWORKS CONSTRUCTED AND OPERATED SOLELY FOR INDUSTRIAL SERVICE[1]

By Claude L. James

Mattoon, Illinois, has the distinction of owning and operating a system of waterworks solely for commercial purposes. The water for domestic uses and fire protection is furnished by a private corporation, the supply being taken from the glacial drift at a depth of about 60 feet. For purity and wholesomeness the water furnished by the private corporation is seldom excelled. The quantity obtainable from this source, however, was found to be insufficient to supply the domestic and commercial needs of the city. In order to have enough water to supply all demands of the railroads and other large consumers, the citizens of Mattoon organized early in 1907 a stock company to construct a waterworks system of sufficient capacity to meet all demands made by the large consumers. The capital stock of $105,000 was fully subscribed by the citizens of Mattoon after a canvass of less than ten days. Before the work of construction was begun, the city entered into a contract with the Mattoon Reservoir Company, agreeing to purchase the system after its completion and the company agreed to sell the plant to the city at the actual cost of construction plus 5 per cent interest. This agreement was carried out, and the city finished paying for the plant in four years time.

After the plant was fully completed a certified bill of cost was made setting out in detail the exact cost of the entire system including all dams, reservoirs, lands, engineering and supervision of construction. The total cost, including the lands, was $103,000. This amount completed the dams, reservoirs, pumping machinery, two pumping stations, suction pipe, intakes and about 5 miles of force mains between the pumping stations.

The Paradise reservoir was formed by throwing a dam across the Little Wabash valley at a point about 6 miles southwest of Mattoon. This dam is an earthen embankment about 400 feet in length, with

[1] Read before the Illinois Section at Urbana on March 14, 1917.

a top width of 10 feet and a slope on the water side of 3 to 1 and one of 2 to 1 on the down-stream side.

The concrete spillway is a structure 100 feet in width between abutments and was originally 20 feet in height above the stream bed. It was provided with a concrete apron 70 by 100 feet. A concrete heel was constructed 25 feet wide, and on this heel was placed a well compacted earth fill to an elevation within 7 feet of the crest of the spillway. The earth dam was built without a concrete core wall. A gravelly clay found near the site of the dam was used as a core wall. A core of this material about 6 feet in thickness was placed along the center line of the dam to an elevation of about 4 feet above the crest of the spillway. Special care was taken in compacting this clay while it was being placed. As it was impossible to use a roller on this part of the embankment, owing to the tendency of the clay to pick up, the material was compacted by driving horses back and forth as it was being placed. The dam has been free from seepage since it was placed in service. The concrete spillway was founded on very unstable soil; no rock, hardpan or other stable material could be located near the dam site. To overcome this condition Prof. A. N. Talbot gave the design of the dam much careful consideration and finally decided upon what might be called a spread foundation. The structure covered an overall area of 105 by 112 feet. The total cost of the earth dam and concrete spillway was $16,000.

The pipe line between the reservoir pumping station and the main pumping station consists of 12,000 feet of 12-inch cast iron pipe and about 13,000 feet of 18-inch vitrified clay sewer pipe. The water is pumped from Paradise reservoir through the cast iron pipe to the top of Magnet Hill, which has an elevation of 75 feet above the reservoir pumping station and 55 feet above the main pumping station. On account of this fact, together with the fact that the price of cast iron pipe in 1907 was $34 per ton, it was decided by the engineers in charge of the design to use vitrified clay sewer pipe as a gravity conduit from Magnet hill to the main pumping station. The sewer pipe used was the deep-and-wide socket type; the joints were made of jute and 1 to 1 Portland cement mortar. One-half of the joints were made on the bank of the ditch several days prior to the time of laying the pipe in the trench. This pipe was subjected to tests in sections of 1000 feet to determine the amount of leakage. These tests proved very satisfactory, as the leakage was very small.

The water supply is obtained from two distinct water sheds. The Kickapoo area in which the main pumping station is located is composed of about 4000 acres of farm land and is drained by 50 miles or more of drain tile ranging in size from 4 to 30 inches in diameter. The flow from these drains is intercepted by a line of sewer pipe laid by the water company, which conveys the water so intercepted to the service reservoir located at the main pumping station. The Little Wabash water shed above the dam comprises an area of 20 square miles. The surface of the ground in this district is of a fairly uniform slope of about 7 feet to the mile.

The service reservoir is 95 feet in diameter and 30 feet in depth, having a capacity of 1,500,000 gallons and cost $8200, including excavation. Since the beginning of the operation of the plant in the spring of 1908 the Kickapoo area has furnished about 60 per cent of the total amount of water pumped into the distribution system.

The Paradise impounding reservoir had an original capacity of 200,000,000 gallons but the droughts of 1913–14 lowered the water in the reservoir to such an extent that the water board deemed it advisable to increase this capacity to 350,000,000 gallons. This was accomplished in the fall of 1914 by raising the dam 30 inches and by the construction of an auxiliary concrete spillway 150 feet in length at a total cost of $4000. In raising the crest of the spillway, the top surface of the concrete weir was roughened by bush hammering. One-inch holes 18 inches in depth, spaced 6 inches apart and staggered, were drilled, and in these holes were placed $\frac{3}{4}$-inch square twisted rods extended from the bottom of the hole to 26 inches above the top of the old spillway. One-half inch horizontal rods, spaced 8 inches on centers were wired to the vertical rods. At each abutment the concrete was cut out to a depth of 6 inches in order to properly bind the new concrete to the old abutments. In joining the new work to the old a remarkably good joint was made and not the slightest leak can be observed at it.

The reason for the construction of the auxiliary spillway was the fact that if the dam was raised 30 inches without providing some method of reducing the height of flood water, a large area of farm land would be inundated at each flood period. This scheme for reducing the water height has proved very successful, as the extreme flood height since the raising of the dam has at no time reached the level that occurred before the construction of the auxiliary spillway.

The water is pumped from Paradise reservoir to the service reservoir by a 6-inch single-stage contrifugal pump direct-connected to a 50-horse power electric motor. The electric current is obtained from the local public service company. The maximum charge is 2.35 cents per kilowatt-hour with an average charge of 1.9 cents. The average cost of current used in pumping 1000 gallons against a total head of 130 feet is 1.2 cents.

The water rate ranges from 4 cents per 1000 gallons where the monthly consumption exceeds 16,000,000 gallons to 12 cents per 1000 gallons where the consumption does not exceed 100,000 gallons per month.

The plant is managed by a board of four members and usually comprises the most capable citizens. By the ordinance creating the board no city official is allowed to become a member except the mayor, who is ex-officio chairman. The citizens of Mattoon are very proud of this water system, especially those who participated in financing the original proposition. The water department now has a sinking fund containing about $35,000 derived solely from the profits accruing from the operation of the works. In addition to this, betterments and improvements have been made since 1908 amounting to about $30,000, all being paid from the earnings of the plant.

Prof. A. N. Talbot was consulting engineer and the author, the present superintendent, had charge of the construction.

PUBLIC UTILITIES AND PUBLIC HEALTH[1]

By H. E. Barnard

Golden apples no longer load the public utility apple tree. There was once a time when it was the stockholders' delight to gather in the orchard when the board of directors shook the tree, but those days are gone and today it is the exceptional tree that will yield its expected 6 per cent. The fortunate owners of utilities before the days when commission pests and board of health codling moths made the crop of golden fruit hard to grow and most difficult to gather, are, like other orchardists, prone to bewail the sad state which has come upon them and deplore the heavy expense to which they are put to make the tree blossom, to say nothing of the continual struggle to mature the crop after the fruit has set. But, coming out of the orchard, where metaphors may please but fail to enlighten, we find the public utility of today charged with a double duty. It must render service to those whom it presumes to serve. It should also return a fair profit to its owners. And the first duty is the greater.

I shall confine my analysis to the public utilities known as water companies, and I shall make no distinction between companies owned by stockholders in distant cities or municipal plants owned by the consumers. In each case the duty to the public is the same. If in the fulfillment of that duty the stockholder suffers he is entitled to sympathy, perhaps commiseration for the lack of judgment that allowed him to invest his money in a business where the first and last concern is service to the purchaser of the product.

Thirty years ago the water company worked under different conditions. Public opinion was uncrystallized and courts interpreted the law with no idea of what we now call social justice.

Then the consumer drank water at his peril; today he drinks with the assurance that if he is injured by his indulgence the water company will compensate him for his illness or his family for his death. There is no question as to the law or the interpretation of the courts. It is an accepted principle that when a man engages in the business

[1] Read before the Illinois Section at Urbana on March 14, 1917.

of selling food he assumes all the risks. If by his act or by the act of his servant a customer is injured, he must be responsible. And the law is the same for the water company as for the butcher. Both are in the business of selling food, for water is a food and so recognized by all federal and state food legislation.

It is unnecessary in this address to point out the danger of drinking polluted water. That has long since been recognized as a first principle of sanitary science. Pure water is safe water. Polluted water is dangerous. Health authorities recognize their responsibility in the protection of water supplies. Many states have special departments which regulate water supplies as public health measures. Indiana exercises a supervisory control over all waters, public and private, through the laboratories of the State Board of Health and has not yet conceded to the Public Utility Commission its complete authority over the operation of water supplies in matters which concern the public health rather than the public pocketbook. Illinois handles the matter even more directly through its State Water Survey, and, the author would add, most successfully. Responsibility is fixed and means are provided so that Dr. Bartow and his assistants may not only safeguard the public but at the same time furnish the water companies with most valuable assistance, and this, in the author's opinion, is the ideal organization. An efficient police department is worth while, but constructive work that in the end makes the policeman unnecessary is far more desirable.

And yet in spite of law, court decisions and plain horse sense, there are still in operation water companies that conceive their purpose in life to be pushing water through pipes and, that accomplished, they claim their reward. In Indiana, and it may be true in Illinois, there are superintendents of water companies on the salary list but not on the job who, so far as can be learned, have never heard of the rights of the people to pure as well as fluid water and who also regard the fire chief as a much more important personage than the health officer. Fire protection is a function of a water supply. No doubt many towns would still be using wells if the fire hazard did not compel the laying of water mains and the installation of hydrants. But as soon as the tap is turned on in the kitchen the situation changes, and the primary duty of the water company is not to maintain pressure for fire protection but to pump food through its line, and above all, food that is sure to promote health and not to breed disease.

Now then, where are we? We are up against the incontrovertible fact that a water supply, save in a few unusual instances, is an object of suspicion. There may have been a time when surface waters were pure, when our lakes and ponds were uncontaminated by the wastes of civilization. But that time is long past, no stream is so remote that it may not be polluted, no pond so protected that it is surely safe. The water company must operate a plant to make unsafe waters dependable in addition to the pumps which were once, and unfortunately still are, the superintendent's chief concern.

From the public health standpoint, and the author maintains that is the all-vital consideration, fire protection is a secondary and minor function. And yet last July an engineer for a water company in an Indiana city, when confronted by the demand of the fire chief for more water with which to fight a serious fire, turned into his mains the water of a grossly polluted river. After such criminal action, for it was criminal in the same degree as the act of the guard who throws open the city gates at the demand of the assaulting enemy, the officials called upon the state board of health for advice.

What we want, what we must have, is a trained, responsible man in every water company who will seek advice first and act only after the community knows what confronts it. The author is not sure that the disaster of fire is ever as menacing as the scourge by typhoid, or that there can arise a situation in which a water company will be justified in filling its mains with polluted water, but if such an occasion should confront the water company its first act should be to protect health by complete warnings.

As water companies have assumed the responsibility of pumping pure water, they have added additional defenses against bacterial contamination. Filter beds, no matter how efficient, are supplemented by chemical treatment. The almost universal use of hypochlorite, or chlorine, protects the consumer and breeds in him a faith that is the greatest asset of the water company. But chemical treatment is only efficacious when it is working. When no chlorine is entering the water the supply is as dangerous as before the plant was installed, indeed, it is more dangerous because the faith of the consumer has stopped his practice of boiling his drinking water.

Within a year four Indiana plants operating chemical purification installations have on at least one occasion stopped the chemical purification for a period of from two days to two weeks and without advising either the health officer or the consumers of their action.

The reason given was, "We ran out of chemicals and it was several days before we could get more." So far as the author is informed, the impossible, inefficient, untrustworthy men who ran out of supplies are still on the job, or at least as much on the job as the sentinel who sleeps at his post can ever be. As damnable as was the neglect of the individual which might have sent typhoid raging through the city, the failure of the "higher-ups," the city officials, the officers of the water company, to appreciate the blood guilt of the subordinate demands even more caustic criticism.

All the consumer got, all the health officer received, was excuses. "The orders were in but the hypochlorite didn't come." "We didn't realize how short we were." "No trouble came of it anyway, so what's the howl about?" When an officer of the armies that are sowing the fields of Europe with mangled men gives his superior such an excuse for failure, he knows that the only answer will be that of the firing squad that hustles him into his freshly dug grave. The responsibility of the man at the pumping station is as grave as that of the officer on the battle field; graver, for the lives of hundreds and thousands are in his keeping, not alone in a single battle but night and day, every day in the year.

Now this is an admitted fact. There are no "maybe's" or "perhapses" to the premise. When a water company sells water it guarantees that water to be pure, it pledges itself to know that it is pure. Why, then, does every political change, petty shifting of civic responsibility from incompetent shoulders to others more or less incompetent, mean a change at the water works? Why does the job go to some party follower or relative of the mayor, who wouldn't recognize a colon bacillus if it was as big as a toad and whose only idea of service is the record of the pressure gauge. We, the consumers, are asleep; we, the health officers, are bound by precedent or fearful of oversteppping our authority; we, the press, are partisan and indifferent; we, the chambers of commerce, are sworn to do nothing that will arouse fear or hurt business.

Paraphrasing Patrick Henry: The price of safety is eternal vigilance. Consumer, health officer and every civic organization, realizing that pure water is not only a personal right but a public asset, must work together to put the water works into the hands of trained, dependable experts, and having chosen wisely, to demand full service. Not until then can we count on the public utility in the form of the water company to render dependable service in protecting the public health.

DISCUSSION

CHAS. R. HENDERSON: The speaker wishes to ask the author to consent to changing the word "company" as it appears in the paper to "department." The speaker fears that in the minds of laymen and in case the paper is printed in non-technical journals, the word "company" may be misleading, even though it is intended to describe a municipal water works as well as a water works owned by a private company. The word "department" would seem to me to be more in accordance with custom if the author will consider the suggestion.

WHAT THE ILLINOIS SECTION HAS DONE AND CAN DO[1]

By Paul Hansen

In the endeavor to find something appropriate to say as chairman of the Illinois Section of the American Water Works Association, it occurred to me that I might get some inspiration from a retrospect of the work that our organization has already performed. A count of the papers that have been presented showed that in all there are about 207, including those on the program for the present meeting. These papers cover a great variety of water works subjects and some that are not strictly water works subjects. An analysis of the number of papers devoted to each subject should prove interesting as an indication of our trend as an organization. It was not possible to classify the papers accurately according to subjects, because many of them dealt with or touched upon several different topics, but it was possible, in a general way, to divide the papers into a number of different groups.

By far the largest group is that comprising papers on water purification. There was a total of 45 papers almost equally divided between filtration and sterilization, 23 of the former and 22 of the latter. The next largest group, comprising 23 papers, is one closely allied to water purification, namely, that group which comprises papers on chemicals, chemistry and analytical methods.

When one considers that in the State of Illinois, there are about 340 water works and only about 36 furnish purified supplies and perhaps 25 more furnish water which should be purified by some means or other, it would seem that possibly a disproportionate amount of attention is being devoted to this special phase of the water works field. The intimate relation between public water supplies and the public health is so great that the necessity of delivering to the public a safe water supply cannot be over emphasized, but we must also take into consideration that if our organization is to be of advantage to the greatest number of water works men, it is important that our efforts be so balanced as to interest all.

[1] Annual address of the Section Chairman, delivered before the Illinois Section at Urbana on March 13, 1917.

There were 15 papers that could not be classified under any special subject and had to be termed "general." They related for the most part to the development and management of water works as a whole. These papers are very valuable and interesting, but, generally speaking, it is believed that they could be made of greater value if their substance were directed to one main theme, thereby rendering them more suitable for reference.

Water pollution is represented by 14 papers, most of which have only an indirect relation to water supply problems. A number of them describe interesting and unusual ways in which water supplies may become contaminated, but most relate to nuisances and other evil results from stream pollution.

Papers on ground waters and ground water development comprise a group of 13, practically all of which are very interesting and valuable. It would seem that in view of the great preponderance of ground water supplies in Illinois and also in view of some of the peculiar and unusual characteristics of Illinois ground water resources, that problems relating to ground waters would command much greater attention.

The next group of considerable size is one relating to pumps and pumping stations. This group also seems disproportionately small when one considers that the operation of pumps and the management of pumping stations constitute one of the very large items of expense in water works maintenance. In view of the dilapidated condition of the great majority of pumping stations in small communities, the desirability for a larger number of papers relating to pumps and pumping stations becomes all the more apparent.

Eight papers are devoted to state supervision and regulation of water supplies, covering the matters of accounting, service, rates and sanitary quality. In view of the large amount of attention that has been given to the control of public utilities in recent years, it seems somewhat strange that there have not been more papers on this subject, but perhaps the deficiency is met by the great amount of attention given the subject by our national organization.

Water softening has to its credit 7 papers, which is a goodly number. However, it is probable that the water works literature will be augmented by much more material on water softening within the next ten to fifteen years, as water softening seems to be the logical next step in water refinement.

Pipes and piping receive attention from 6 papers, which is a

rather small number, considering the important factor in water works practice represented by the piping system.

Five papers each were devoted to iron removal, railway water supplies, reservoirs and dams, water waste and leakage. Four papers each, were devoted to hydraulics, fire protection and meters. Three papers each were devoted to water rates, sewage disposal and the business management of water works. Two papers each were devoted to house services, anchor ice and valuation of water works utilities. One paper each was devoted to franchises, swimming pools, color removal, hydrotherapy, vital statistics, flood control, materials of construction used in water works practice, publicity in connection with water works, and use of water in industries.

Among the subjects that received this scanty attention from our section, there are some that are deserving of much more recognition. Notable among these are, business management of water works, publicity in connection with water works, house services, water rates, reservoirs and dams, water waste and leakage, and meters. The importance of these subjects is self-evident to men of the water works fraternity, but it may not be amiss to dwell on one or two of these items.

In my own experience in traveling about the state I have been especially impressed with the exceedingly slip-shod manner in which water works properties are operated, especially in the smaller communities. Some of these water works constitute a great menace to public health, while all of them give poor service and the great majority of them are costly to operate. The difficulty lies, especially in the small municipally owned properties, in securing competent supervision. This is no reflection upon the character of these communities or the people in them. The trouble is due to the fact that a small community cannot afford to employ the high grade men that are engaged in operating the larger installations, and then, too, there is general lack of appreciation in the smaller communities of good business management and efficiency in operation. To me it appears that the relief must come from the outside and it must be afforded at a small cost. State agencies can do much in an educative way, and in the cases of water supply that, through improper operation, may constitute a menace to public health, it is possible for the state to demand and enforce at least enough good operation to insure the water being of safe quality. But the most readily available assistance is that of practicing engineers at present

engaged in the design and construction of water works. They should extend their function to cover the field of operation and should actively offer their services on an annual fee basis for supervising the operation of water works plants.

This service can take the form perhaps of quarterly visits to the plant and the receipt at least monthly of operating records made out on forms to be prescribed by the engineers. Such service will not only greatly improve operation, but it will improve design as well. Under present circumstances, an engineer has little opportunity to ascertain how his designs work out in practice, unless he is willing to revisit his works at his own expense. The arrangement will also be of material financial advantage to engineers in that it will assure them of a steady income for taking care of over-head expenses.

Water works properties are far behind other utilities in the matter of publicity. In 1915, there was a very excellent paper read before this section on the practical value of publicity to water works men by S. C. Hadden, and I regret to note that his very excellent suggestions did not arouse more active interest on the part of our membership. Among the methods whereby water works publicity can be conducted, he mentioned, first, the preparation of readable annual reports; second, the preparation of pamphlets on special subjects for distribution among water takers; third, the preparation of reading matter for publication in local papers; fourth, formal advertising; fifth, circularizing lists of prospective customers; sixth, making addresses before gatherings of people in the locality served; seventh, holding departmental meetings; eighth, the use of hand bills and posters; and ninth, identification and participation in the activities of associations of water works men. In the matter of publicity, much greater advance has been made in connection with other public utilities than with water works. The attitude of the water works men has been too often that the public must have water and, therefore, the business does not require boosting. While this is in a measure true, there is no doubt but what a better feeling could be developed between the public and the water works industry than now exists. Further, a much greater use of water could be stimulated just as there is now being stimulated a greater use of electricity and gas, commodities which are in no sense necessities, as is water, and finally it is of very great importance to the public health where water works exist and where the water is of pure quality that the public supply be used in preference to private wells which are almost certain to become contaminated in urban districts.

Many of the experiences of water works men in connection with the subjects that I have mentioned as not having received the attention their importance demands are of a character that do not seem to warrant the preparation of a paper, yet could be most advantageously taken up as an informal discussion. The need for such informal discussions is amply attested by the popularity that round-table discussions have recently developed. Accordingly, I think that a much greater proportion of our meetings should be given up to round-table discussions on announced subjects and instead of striving to secure a large number of papers, we should bend our efforts to securing a large number of very short discussions on a variety of timely topics. Moreover, it is particularly appropriate that such discussions should take place in the sectional meetings of the American Water Works Association, because these sections have been formed for the very purpose of promoting a closer acquaintanceship in their respective localities and for affording an opportunity for greater freedom of expression than is possible at the national conventions.

STERILIZATION BY LIQUID CHLORINE AND HYPOCHLORITE OF LIME

By M. S. Dutton

In the following comparison of the use of liquid chlorine and hypochlorite of lime, the data apply in particular to one filtration plant, a description of which follows, and in general to other installations where this means of sterilization are in use. The plant was put in operation the latter part of January, 1915. The water supply is taken from a small stream and is at all times subject to contamination by sewage and waste water from coal mines. The supply is pumped from the stream by two triplex plunger pumps at a low-lift pump house, three-quarters of a mile to the filtration plant. It flows through a weir box, mixing chamber, two coagulating basins, three filter units, and thence to a clear well under the filter building. From this clear well the water flows to another larger clear well from which the service pumps draw their supply. A very satisfactory place for the application of either liquid chlorine or hypochlorite of lime has been the 8-inch pipe connecting the two clear wells. This place for the application has eliminated any loss of chlorine due to direct contact with the air and splashing water. The plant has a capacity of 900,000 gallons per day and was constructed with tanks for appying hypochlorite. Although the use of liquid chlorine was considered, the cost of an apparatus for the application of the chlorine seemed excessive in view of the fact, too, that the apparatus had not been perfected to its present state. In February, 1916, it was decided to purchase a chlorinator and since that time all former troubles connected with the application of hypochlorite have been eliminated.

The installation consists of a chlorinator of the type which supplies a solution of chlorine and water to the supply. Liquid chlorine is received in drums containing 105 pounds and as it feeds to the chlorinator, expanding to a gas, it is mixed with water and fed through a hard rubber hose to the point of application. The manufacturers of this apparatus claim many advantages over the use of hypochlorite, the most important of which are:

Cost analysis of liquid chlorine and hypochlorite

DATE 1916	PUMPAGE	HYPO USED	HYPO USED PER MILLION GALLONS	COST OF HYPO PER MILLION GALLONS	DATE 1916	PUMPAGE	LIQUID CHLORINE USED	LIQUID CHLORINE USED PER MILLION GALLONS	COST OF LIQUID CHLORINE PER MILLION GALLONS
	gallons	*pounds*				*gallons*	*pounds*		
January					June				
1	276,500	5.2	18.8	$1.32	1	282,000	0.39	1.38	$0.28
2	237,500	3.8	16.0	1.12	2	286,000	0.40	1.40	0.28
3	244,000	3.5	14.3	1.00	3	294,000	0.42	1.43	0.29
4	277,000	4.6	16.6	1.16	4	263,000	0.37	1.41	0.28
5	208,000	3.3	15.9	1.11	5	337,500	0.48	1.42	0.28
6	216,500	3.6	16.6	1.16	6	289,500	0.41	1.42	0.28
7	239,500	3.7	15.5	1.08	7	331,000	0.47	1.42	0.28
8	252,600	5.8	23.0	1.61	8	239,000	0.34	1.42	0.28
9	273,000	5.5	20.2	1.41	9	308,000	0.43	1.40	0.28
10	369,000	5.0	13.6	0.96	10	212,500	0.30	1.41	0.28
11	340,000	6.1	17.9	1.25	11	202,000	0.28	1.39	0.28
12	262,500	3.8	14.5	1.01	12	221,500	0.31	1.40	0.28
13	354,000	4.6	13.0	0.91	13	192,000	0.27	1.41	0.28
14	600,000	11.1	18.5	1.29	14	308,000	0.43	1.40	0.28
15	475,000	8.7	18.3	1.28	15	248,000	0.35	1.41	0.28
16	284,000	10.4	36.6	2.56	16	233,000	0.33	1.42	0.28
17	444,000	6.7	15.1	1.06	17	216,500	0.30	1.39	0.28
18	433,000	5.9	13.6	0.96	18	300,000	0.42	1.40	0.28
19	416,000	5.0	12.0	0.84	19	402,000	0.57	1.42	0.28
20	433,000	4.3	9.9	0.69	20	244,000	0.34	1.39	0.28
21	481,000	4.6	9.6	0.67	21	175,000	0.25	1.43	0.29
22	378,000	5.4	14.3	1.00	22	336,000	0.47	1.40	0.28
23	288,000	4.8	16.7	1.17	23	202,000	0.28	1.39	0.28
24	283,000	4.5	15.9	1.11	24	214,000	0.30	1.40	0.28
25	288,000	4.5	15.6	1.09	25	380,000	0.54	1.42	0.28
26	359,000	4.3	12.0	0.84	26	352,000	0.50	1.42	0.28
27	221,000	2.9	13.1	0.92	27	404,000	0.57	1.41	0.28
28	423,000	2.7	6.4	0.45	28	352,000	0.49	1.39	0.28
29	316,200	3.3	10.4	0.73	29	361,000	0.53	1.47	0.29
30	222,400	2.8	12.6	0.88	30	316,000	0.46	1.47	0.29
31	336,000	2.8	8.3	0.58					
Total	10,230,700	153.2				8,501,500	12.0		
Average	330,000	4.9	15.3	1.07		283,380	0.40	1.41	0.28

Note: Cost of hypochlorite, 7 cents per pound; cost of liquid chlorine, 20 cents per pound.

1. A more accurate means of control.
2. An elimination of dirt and the odor of chlorine.
3. An appreciable decrease in labor.
4. A means of storing the chemical so as to eliminate any decrease in strength.
5. An elimination of chlorine taste in the treated water which is so often found where hypochlorite is used.
6. A decrease in the cost of chemical required.

After having this apparatus in service a little over a year, to date, the first five claims for this method of sterilization have been realized and the accompanying figures substantiate the claim that a material decrease has been made in the cost of chemicals.

Summarizing briefly the tabulated cost analysis, the cost of hypochlorite being considered as 7 cents per pound and the cost of liquid chlorine as 20 cents per pound, the average cost of sterilization by means of hypochlorite was $1.07 per million gallons of water treated and $0.28 per million gallons when using liquid chlorine. The saving, then, in chemicals alone amounted to $0.79 per million gallons of water treated.

In summarizing the results of sterilization, the average reduction in bacteria count of daily plates made on nutrient agar incubated at 37°C. for 24 hours was as follows: In January, when hypochlorite was used, the average raw water count was 6300 and the filtered water count was 15. In June, when liquid chlorine was used, the average raw water count was 7980 and the filtered water count was 11. In B. coli tests, the raw water shows 100 per cent + for both months, while the filtered water shows 1.60 per cent + in January and 0.0 per cent + in June. These results were obtained by means of 1 cc. samples in all cases. Also note that while hypochlorite was used 0.46 p.p.m. of chlorine was applied and while using liquid chlorine 0.22 p.p.m. of chlorine was applied.

As previously stated, and as brought out in subsequent operation reports, sterilization by liquid chlorine has eliminated numerous former troubles and has at all times affected a saving in the cost of chemicals required.

TREATMENT OF PUBLIC WATER SUPPLIES FROM LAKE MICHIGAN[1]

By Samuel A. Greeley[2]

Extent. Probably no bodies of water are used more extensively for water supply purposes than the Great Lakes. Of these, Lake Michigan ranks well to the front in the actual population which it supplies with water. From the large population centering about Chicago, amounting to about 3,000,000 people, down to the smaller villages of a few hundred people, all sizes and varieties of public and private water supplies are taken from the lake. A rough estimate of the total population supplied by water from Lake Michigan is 4,000,000 people. It is therefore of importance and interest to consider briefly the treatment of these public water supplies.

Source. The supply is drawn from the lake at varying depths of water and distances from shore. In the smaller intakes this is usually from a point near the bottom of the lake. In the larger intakes the water may be taken from mid-depth to the surface. The intakes farthest from shore are those at Chicago, the farthest out being 4 miles. The ports of the Chicago intakes are at a depth of about 33 feet. The intake at Milwaukee extends about 8000 feet from the shore to a crib in water 60 feet deep. The shallower intakes are those near the southern end of the lake. At East Chicago, Ind., the depth of water at the intake is only 20 feet. At Gary, Ind., an intake with a length of 17,000 feet reaches water 44 feet deep. Thus a considerable variation in the character of the water supply results from the location of the intakes.

Pollution. The near shore waters of Lake Michigan in the vicinity of the smaller cities are polluted by the sewage from over 1,000,000 people as well as from a large number of industries. Shipping also adds pollution. Only the sewage of Chicago and some of the adjacent towns within the Sanitary District of Chicago is now diverted from Lake Michigan. The balance of the sewage is discharged

[1] Read before the Illinois Section at Urbana on March 14, 1917.

[2] Hydraulic and Sanitary Engineer, 64 West Randolph Street, Chicago.

directly into the lake, for the most part without treatment. Where sewage treatment plants have been built, operation results chiefly in a reduction of the suspended matter and the oxygen requirement of the sewage rather than in bacterial purification.

This pollution, entering near the shore, is mixed with the lake water and carried considerable distances by natural agencies, principally the wind, which induces drifts in the lake. It is estimated by Judson and Evans that sewage pollution may be carried in this way as far as 20 miles. Where the volume of sewage is large, active pollution has been traced at Milwaukee a distance of about 5 miles from the main sewer outlet. Floating ice, wind driven, may also carry sewage pollution from points of greater intensity to points where the pollution is supposed to be slight. A moderate typhoid outbreak at Marquette, Mich., as well as one at Erie, Pa., were attributed by some to the movement of ice, carrying sewage pollution from the sewer outlet to the water supply intake. Actual chemical and bacterial analyses of the lake water by the Lake Michigan Water Commission and the Indiana State Board of Health have shown that sewage pollution at some points between Waukegan, Ill., and Gary, Ind., extends at times 5 miles from shore.

In addition to the sewage from the municipal sewer outlets and the industrial plants, a substantial amount of pollution comes from (1) private houses and clubs so located that they cannot enter a general sewer system, (2) from overflows on combined sewer systems, and (3) from surface run-off during times of storms. Moreover some pollution comes from ships and pleasure boats. This may be particularly dangerous where the course of vessels passes directly by the intakes, as at the entrance to the Chicago River.

The pollution varies in strength from season to season and even from day to day. Under certain directions of the wind, the pollution is greater at some points than at others, while a shift in the wind may entirely reverse the condition. During periods of calm some sedimentation occurs, particularly immediately near shore. During periods of storm, the sludge is stirred up from the lake bottom, increasing the pollution. The effect of ice in winter also bears important relation to the extent and strength of the pollution at particular points.

Intakes. The relation of intakes to the purity of the water supplies from Lake Michigan is a factor which has long been recognized. This has been the case in Chicago where intakes have been

extended from year to year as the population has increased. It is undoubtedly true that for most of the time, intakes farther from shore tap a less polluted water, and particularly a water which varies less in quality from day to day. The relative importance of new or extended intakes as compared with water filtration and other methods of treatment has been frequently discussed. The tendency at present leans toward the water filtration plant. In this connection two points should be emphasized. In the first place in actual operation it is found that anchor ice frequently clogs the outer end of the intakes, thus forcing the use of emergency near-shore intakes. This has occurred at East Chicago, Ind., North Chicago, Evanston and Waukegan, Ill., and elsewhere. When this is done, an additional load of pollution results which may seriously disturb the operation of a chlorinating plant if used alone and thus remove the safeguard relied upon by the consumers.

In the second place, breaks and leaks in the intake may occur near shore without the knowledge of the water works superintendent. Thus a sudden and unexpected pollution results which is more likely to dangerously upset the routine treatment of the water. This occurred at Evanston, Ill., in 1912, causing an outbreak of typhoid. It has also occurred at Gladstone, Mich. Thus the value of intakes, particularly on the smaller installations, must be viewed with certain definite qualifications.

Need for treatment. The need for treating public water supplies taken from Lake Michigan is obvious and is recognized at practically all places. The most popular treatment is with calcium hypochlorite or liquid chlorine. This of course is a cheap method which can be quickly installed, and is of great value as an emergency project. A more certain method of treatment is by rapid filtration, a method which is now being used extensively. Rapid sand filter plants are in successful operation at Escanaba, Mich., South Milwaukee, Wis., Evanston, Ill., and probably at one or two other towns. Filters of the closed pressure type are used at Lake Forest, Kennilworth and Rogers Park, Ill. A slow sand filter has been in use at the U. S. Naval Training Station but is now being changed to a rapid filter.

By far the largest amount of water, however, is treated with hypochlorite of lime or liquid chlorine alone. The cities of Chicago, Milwaukee, Waukegan, Whiting, Hammond, East Chicago, Michigan City, etc., treat all of their public water supplies from Lake Michigan in this way.

Chlorine treatment. Treatment of water supplies from Lake Michigan with hypochlorite of lime or liquid chlorine has the distinct advantages of low first cost, low cost of operation and the rapidity of installation. These advantages are all characteristics of an emergency treatment. There are a number of disadvantages which it is worth while to note. As already stated, the quality of Lake Michigan water is subject to frequent and rapid variations as regards the content of suspended organic matter and the strength of bacterial pollution. Thus it is necessary at frequent intervals to adjust the amount of chlorine added to the water.

TABLE 1

Pounds of calcium hypochlorite used per million gallons at Waukegan, Ill.

DATE, 1916	12 TO 4 A. M.	4 TO 7 A. M.	7 TO 12 NOON	12 TO 3 P. M.	3 TO 8 P. M.	8 TO 12 NIGHT
February 6.....	9.45	10.96	22.37	9.65	14.60	16.71
February 3.....	7.89	7.55	7.07	7.50	8.07	6.30
February 7.....	24.23	11.66	22.20	20.11	10.08	10.71
February 8.....	7.90	5.55	13.49	9.69	13.20	13.08
February 9.....	7.88	7.58	13.67	17.58	9.16	5.60
February 28.....	15.07	13.04	16.66	15.03	15.19	12.85
March 18.....	11.77	10.00	10.11	9.10	9.84	9.27
March 20.....	8.24	7.57	7.44	7.54	7.90	7.65
March 22.....	8.29	8.35	8.13	8.20	9.59	8.20
March 23.....	6.48	8.74	8.35	7.74	7.62	7.27

Table 1 indicates variations in the amount of calcium hypochlorite added to the water supply at Waukegan. The water is tested at intervals of approximately four hours by the starch iodine test under the supervision of W. J. Allen, superintendent of the water works. It appears from this table, that between the morning of February 6, 1916, and the afternoon of the same day, the required amount of hypochlorite changed from 22.37 to 9.55 pounds per million gallons. The required amount then increased in the late afternoon to 14.6 pounds per million gallons. It is apparent that if the amount of chlorine added to the water were not adjusted to the need, an under-dose or over-dose would result. The under-dose would probably not thoroughly disinfect the water and an over-dose would produce objectionable tastes and odors.

To obviate the undesirable effects of an over-dose, a few water works superintendents have resorted to the use of sodium thio-

sulphate, which neutralizes the excess amount of chlorine. This has been tried at Milwaukee, Wis., and at East Chicago, Ind. The danger of this method is that the thiosulphate may be applied at too short a time interval after the application of the chlorine, thus neutralizing the chlorine before it has accomplished the disinfection. It appears unsafe under ordinary circumstances to add both chemicals in the same well. The experience at both Milwaukee and East Chicago would indicate that the use of the thiosulphate should be undertaken only as a last resort under the most favorable conditions where time intervals of the magnitude of an hour or more are available.

Other disadvantages of the chlorine treatment are its inability to cope with sudden unsuspected increases in the load of pollution, resulting from the use of emergency intakes near shore or from breaks in the intake pipe. It is also true that occasionally water works plants run out of hypochlorite, particularly under present market conditions, as happened recently at Whiting. Another disadvantage of the hypochlorite treatment is the fact that it is less efficient in cold weather and that consequently over-doses are needed at times during the winter months. The possibility of tastes resulting from the chlorine treatment is, of course, familiar.

Treatment by filtration. The treatment of water supplies by filtration has the decided advantage that it is able to cope with marked variations in the quality of the raw water without reducing the bacterial efficiency. In other words the filtration plant is always on the job, delivering a water of reasonably uniform quality substantially free from pathogenic bacteria. If the filtered water be, in addition, treated with liquid chlorine the greatest safeguard to the consumers results. Small doses of chlorine can then be used, and better results obtained than in treating the raw water. In addition, a filtration plant removes turbidity, microscopic organisms and other impurities. The only disadvantage is, of course, the higher first cost and annual cost for operation. However, the failure of the chlorine treatment to afford thorough protection against typhoid, as demonstrated particularly in the smaller towns around Lake Michigan, has resulted in a wholesome trend towards filtration. Filtration plants have been recommended for East Chicago, Ind., and Wilmette, Winnetka, North Chicago and Waukegan, Ill. A plant is under construction at Kenosha, Wis., and plans are in preparation for a plant at Whiting, Ind.

In the design and construction of filtration plants there are several points which require careful consideration. These are due chiefly to the changes in the quality of the lake water at different points along the shore, and to the effect upon the water of the length of intake and the depth at point of intake. Thus there is a marked variation in the amount of turbidity present requiring removal. It has also been found in practice that at certain points along the lake, substantial growths of microscopic organisms develop, which materially affect the length of runs between washings of rapid sand filters. This point can be handled in properly designed plants by efficient operation.

Of particular importance, however, are the amount and character of pollution from industrial plants. Thus at Whiting, where the author's firm is now making investigations preparatory to the preparation of plans for a filtration plant, the waste from the Standard Oil refineries, though small in amount, imparts a decided oily taste to the water. The purified water should be free from this taste, if popular faith in water filtration is to be maintained. In the same vicinity, also, is the waste from a glucose factory and steel mills. At the steel mills the quench water from the coke ovens imparts a so-called creosote taste to the water which resembles the taste resulting from an over-dose of chlorine. Further north along the shore, the discharge from tanneries and other industries is important. Thus each particular water supply along Lake Michigan requires special consideration if satisfactory results are to be obtained.

Result of treatment. It is particularly pertinent at the present time to record a few results of these two methods of treating public water supplies from Lake Michigan. At present there is an outbreak of typhoid at North Chicago and Waukegan, Ill. Investigations by the State Board of Health indicate that the typhoid at North Chicago has resulted because the liquid chlorine apparatus failed to operate for a period during December, 1916. It is uncertain yet as to whether the typhoid at Waukegan did not result from an overload on the hypochlorite plant, due to the use of the near-shore intake during anchor ice troubles at the regular intake. At East Chicago, Ind., during the winter of 1916 something like 100 cases of typhoid fever occurred in a population of less than 30,000. The water supply was treated with liquid chlorine followed almost immediately by the addition of sodium thiosulphate. A less extensive outbreak occurred during the present winter, although the treat-

ment with liquid chlorine continued and the addition of thiosulphate was stopped. The engineering press has reported an outbreak of typhoid fever and intestinal diseases which occurred in Milwaukee during February and March, 1916. This water supply was treated with liquid chlorine. A special investigation was made by the epidemiologist of the Milwaukee Health Department. The author quotes as follows from his report to the Health Commissioner, Dr. Geo. C. Ruhland:

> It is our opinion that this outbreak of typhoid fever was caused by two factors.
>
> The first factor was the unusual concentration of sewage about the water intake during the week of January 19–26, due to the coincident warm weather resulting in an unusual flow of surface washing, combined with the usual sewage discharge from the city, and which was driven across the intake by prevailing southerly winds, thus resulting in an increased sewage concentration of the city water.
>
> The second factor was the failure of the chlorination process to care for this increased pollution with the same efficiency that had been the case previously. It was, therefore, in our opinion a water-borne infection, based on the following reasons:
>
> *First.* The explosive outbreak of diarrheal disease, followed in from three to four weeks by a tremendous increase in the typhoid morbidity.
>
> *Second.* The distribution of the cases in all portions of the city reached by the general water supply, and their absence in the community lacking this supply.
>
> *Third.* Its distribution regardless of the sanitary condition of the homes, location of toilets, or general sanitation.
>
> *Fourth.* The absence of any evidence pointing to occupation, or age. It affected all ages, and the distribution among the sexes was approximately equal. It was more frequent in small than in large families.
>
> *Fifth.* Contact cases could not be traced until the outbreak was at its height or was on the decline.
>
> *Sixth.* Absolutely no evidence was discovered pointing to infected milk or other food supplies which were in common use by the entire population of the city, or even by those infected.
>
> *Seventh.* The prompt subsidence of the outbreak as soon as boiling of the water began to be generally practiced.

Tendencies. The facts and experiences outlined point strongly to the desirability of installing water filtration plants for the treatment of the various water supplies derived from Lake Michigan. This tendency has developed into definite recommendation whereever the matter has been taken up along engineering lines, particularly in the smaller communities. Thus, Alvord, Eddy and Ful-

ler in their report to the Trustees of the North Shore Sanitary District, make the following statement:

Purity of water supplies. As has already been pointed out, Lake Michigan is year by year becoming less reliable as a source of pure water supply, from causes both within and without the district. While this is due in part to the discharge of untreated sewage, in part to the discharge of incompletely purified effluents, and in part to the natural run-off of storm water from the more or less thickly populated drainage area within and without the District, and while much may be accomplished in preventing contamination by untreated and partially treated sewage, it is not practicable to prevent all contamination due to storm water.

The District must therefore rely upon purification to assure the purity of supplies taken from the Lake. Such purification plants must be carefully and skillfully operated.

As the burden of contamination increases, sewage treatment can be relied upon to reduce the load upon the water purification plants sufficiently to assure their ability to produce a safe water. Such sewage treatment plants, however, must be carefully and skillfully operated. With such operation of both sewage and water purification plants, two barriers to the contamination of the water supply will be established.

Dabney H. Maury in reporting upon the water supply at Wilmette advised the installation of a rapid sand filter plant as the best means of securing safe and wholesome water. The author's firm, in work at East Chicago and Whiting, Ind., and at Evanston, Winnetka, and North Chicago, Ill., advised the installation of water filtration plants. At Evanston, the filter plant has been in operation since 1914, with the result that there were only 7 cases of typhoid in 1914, 7 in 1915 and no cases in 1916. At Whiting, Ind., plans for a filtration plant are now being prepared. At East Chicago, the installation of a filtration plant by the private water company is up before the Public Service Commission of Indiana. The plans are also being prepared by Metcalf and Eddy.

Summary. In the opinion of the author, eventually all public water supplies from Lake Michigan will be filtered. In the meantime, where reliance must be placed on treatment with chlorine, careful, expert supervision should be provided and routine analyses made at frequent intervals during the day to regulate the amount of chlorine applied to the varying conditions.

SOME TESTS ON MOTOR-DRIVEN DEEP-WELL PUMPS[1]

By I. W. Fisk and P. S. Biegler

The data presented in this paper have been collected at the suggestion of Mr. F. C. Amsbary, manager of the Champaign & Urbana Water Company. Originally, the object of the work was to correct the low power factor of the power plant load, but other problems arose in the course of this investigation, some of which are discussed below.

A preliminary test showed that the generator in the plant was overloaded, due to poor power factor of the load. This low power factor was undoubtedly due to the fact that the 10-horse-power induction motors driving the well pumps were, as a rule, too large for their loads. They have been replaced by smaller motors as rapidly as possible, thus raising the power factor of the system and relieving the generating apparatus.

Table 1 is a summary of data taken March 7, 1917, to show the general condition of the pumping units at the present time. From similar data attempts have been made to formulate some relations between speed of pump and efficiency, length of stroke and efficiency, etc., but mechanical conditions in the pumps and wells apparently play such a large part in the results of the tests that no such relations have been shown from these data. It may be observed that the efficiencies vary all the way from 19 to 61 per cent, the average over-all efficiency being 46.4. The efficiencies are low when the slips are great, indicating poor condition of valves or pistons. The last columns are of considerable interest since they show the improvement in power factor on the electric motor when a 5 horse power motor is used instead of one of 10 horse power. But, for these loads, there is little difference in motor efficiencies, the results slightly favoring the 10 horse power motor. The measurements shown in this table were obtained with a good degree of accuracy, except those of the head. This was obtained on the

[1] Read before a meeting of the Illinois Section at Urbana on March 14, 1917.

TABLE 1

General operating condition of motor-driven deep-well pumping units on March 7, 1917

WELL NO.	TYPE PUMP	SPEED R. P. M.	KW. INPUT	H. P. INPUT	H. P. MOTOR	H. P. INPUT TO PUMP	STROKE PUMP	BORE	NOMINAL DISPLACEMENT	GAL. PER MIN. ACTUAL	NOMINAL GAL./MIN.	NOMINAL SLIP GAL./MIN.	SLIP IN PER CENT	ACTUAL HEAD	ACTUAL W. H. P.	ACTUAL EFF. PER CENT	MOTOR EFFIC.		MOTOR P. F.	
																	5 h. p.	10 h. p.	5 h. p.	10 h. p.
13	A	18.0	4.37	5.85	10.0	5.5	15	7.43	5.65	86.0	101.5	15.5	15.3	156.0	3.38	57.8	84	86	82	69
15	A	23.0	6.04	8.07	10.0	7.1	15	7.43	5.65	97.5	130.0	32.5	25.0	158.2	3.89	48.2		87		76
14	A	25.5	3.94	5.28	10.0	4.5	15	5.75	3.38	66.5	86.2	19.7	22.8	152.2	2.55	48.2	84	86	77	62
12	A	23.7	5.35	7.16	10.0	6.25	15	7.43	5.65	111.0	134.0	23.0	17.2	160.8	4.48	62.6		87		72
11	A	25.0	3.54	4.74	5.0	4.0	15	5.75	3.38	74.5	84.5	10.0	11.8	153.8	2.89	61.0	84	85	74	59
17	A	24.0	3.41	4.56	10.0	3.9	18	5.75	4.06	62.0	97.4	35.4	36.3	151.4	2.36	51.8	84	85	74	58
1	A	18.6	4.27	5.71	5.0	4.8	15	5.75	3.38	51.0	62.9	11.9	18.9	149.4	1.92	33.6	84	86	78	65
2	A	28.5	4.10	5.48	5.0	4.6	15	5.75	3.38	59.0	96.2	37.2	38.7	150.8	2.24	40.8	84	86	77	63
6	A	27.5	4.15	5.55	5.0	4.65	15	5.75	3.38	48.5	93.0	44.5	47.8	148.8	1.82	32.8	84	86	78	64
3	A	28.0	4.45	5.95	5.0	5.0	15	5.75	3.38	83.5	94.7	11.2	11.8	155.2	3.28	60.2	84	86	79	66
19	A	25.3	3.48	4.67	10.0	3.9	15	5.75	3.38	59.4	86.0	26.6	30.4	150.9	2.26	48.4	84	85	74	58
21	A	29.0	4.04	5.42	5.0	4.53	15	5.75	3.38	67.0	98.6	31.6	32.1	152.3	2.57	47.4	84	86	77	63
5	A	23.0	23.58	9.8	10.0	4.08	15	5.75	3.38	40.5	77.8	37.3	47.7	147.5	1.51	31.4	84	85	70	59
16	B	23.1	4.60	6.17	10.0	5.3	18	5.75	4.06	86.0	93.7	7.7	8.2	156.0	3.38	54.8	84	86	80	68
7	C	29.0	4.87	6.52	5.0	5.45	18	5.75	4.06	75.0	117.8	42.8	36.3	153.8	2.91	44.6	84	86	80	68
8	C	26.4	4.71	6.32	5.0	5.28	18	5.75	4.06	45.5	107.5	62.0	57.6	148.1	1.70	26.9	84	86	80	67
18	C	24.0	4.05	5.43	10.0	4.65	18	5.75	4.06	28.0	97.4	69.4	71.1	145.0	1.04	19.2	84	86	78	64
4	C	23.6	4.28	5.74	5.0	4.8	18	5.75	4.06	53.5	95.8	42.3	44.0	149.7	2.02	35.2	84	86	78	65
20	E	21.3	5.20	6.97	7.5	5.96	15	7.43	5.65	106.0	121.0	15.0	12.4	159.8	4.27	61.2		87		72
22	E_c	26.5	5.18	6.95	5.0	5.8	18	5.75	4.06	101.0	108.0	7.0	6.5	158.8	4.04	58.2	84	86	82	71
10	E	29.5	5.05	6.77	10.0	5.9	15	5.75	3.38	81.8	99.8	18.0	18.0	154.3	3.20	47.3	84	86	82	71
Total				124.3						1483.2	2083.8	597.2			57.71	46.4				
average				5.93						70.6	99.2	28.4	28.6		2.75					

assumption that the water level falls off with the rate of pumping water in all wells, following the relation existing between these two quantities in Well 10, as shown in Figure 4, the data for this well having been taken with great care. In other wells the clearance between pump barrel and well casing was too small for the apparatus available for measuring head with the pump in operation.

In testing motor-driven reciprocating pumps considerable difficulty is encountered from the pulsating character of the work done. The current drawn from the line by the driving motor fluctuates rapidly as shown by the oscillogram, Figure 1. The steady quantity

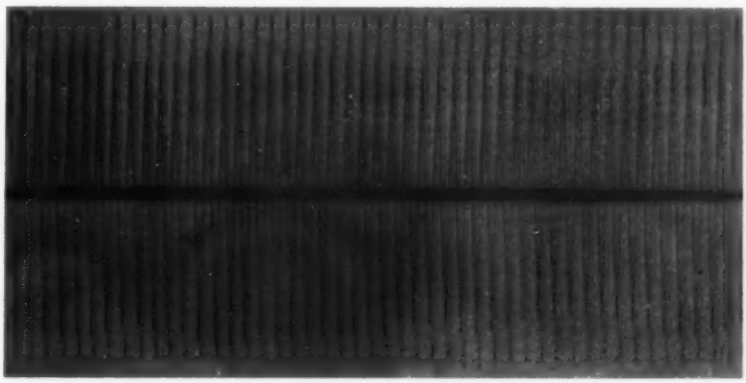


FIG. 1. OSCILLOGRAM OF MOTOR CURRENT

is voltage at the motor terminals, the other curve is the current, which is seen to change rapidly during four-fifths of a second, as shown in the figure. There are no suitable indicating instruments for measuring such fluctuating power or current, but a watthour meter may be used for measuring energy over a given period and from this the average power may be obtained. For all tests discussed in this paper, a rotating standard watthour meter was used to measure the average power required by the motors.

For the measurement of the discharge of the pumps a calibrated bucket was used, as shown in Figure 2. This bucket has holes in the bottom which may be changed according to the rate at which water is delivered by the pump. A glass gage registers the height of water in the bucket and this gage reading may be converted into gallons per minute by reference to Figure 3. By this means any peculiar characteristics of the discharge may be noted which might

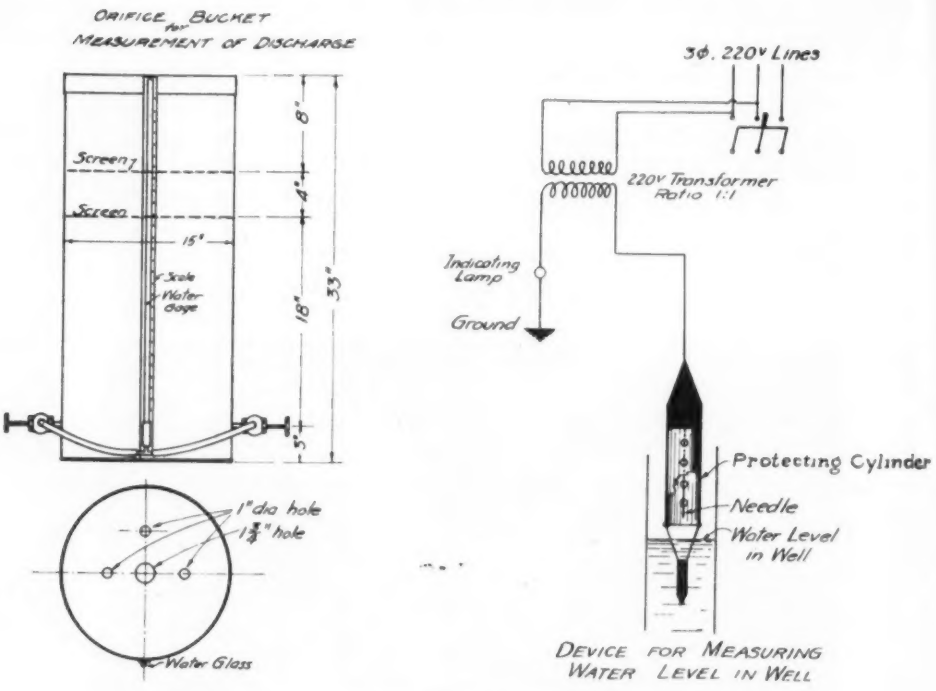


FIG. 2. APPARATUS FOR DETERMINING THE YIELD OF WELLS

pass unobserved if the discharge is measured, for instance, by taking the time required to fill a tank. This device was originally designed by Prof. M. L. Enger, University of Illinois.

Figure 2 also shows apparatus developed by the authors for the measurement of the height of water in the well. It is easily made sensitive to a change in water level of 0.1 inch, the only difficulty being the small space within the well casing into which the device must be lowered to get the water level while the pump is in operation. The device is easily lowered into the well when the pump has been pulled.

From the diagram of connections it is seen that when the end of the circuit being lowered into the well touches the water, a circuit, established through the ground, is complete, lighting the indicating lamp. The use of a transformer in this circuit is simply to avoid grounding one of the service wires. The needle forms one end of the electric circuit and is protected from grounding on the iron

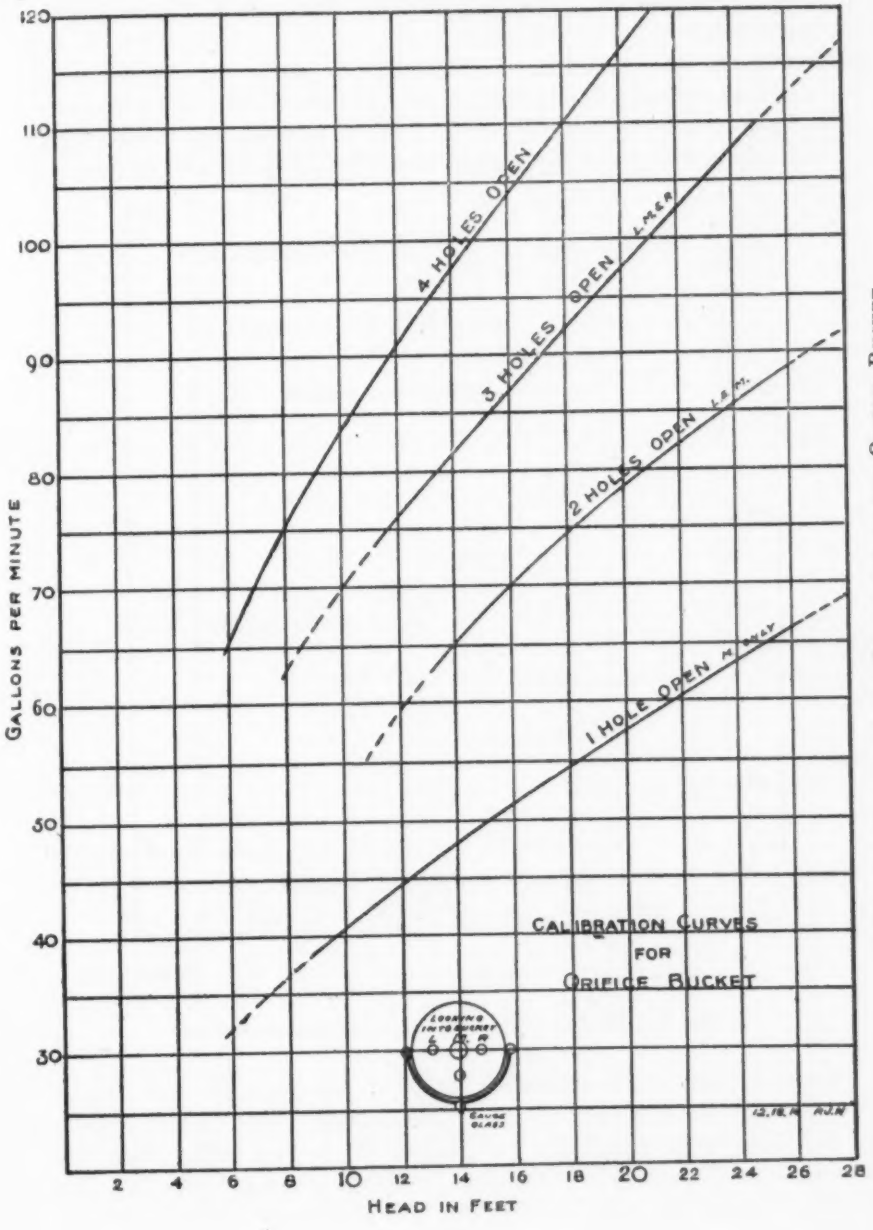


FIG. 3. CURVES OF THE DISCHARGE OF THE ORIFICE BUCKET

pipes by means of a shielding cylinder from which the needle is insulated.

For the reason that it was impossible to get at any definite relations regarding pump speed, water discharged, slip and efficiency from such data as those shown in Table 1, in which the pumps were not all in the same mechanical condition of operation, it became

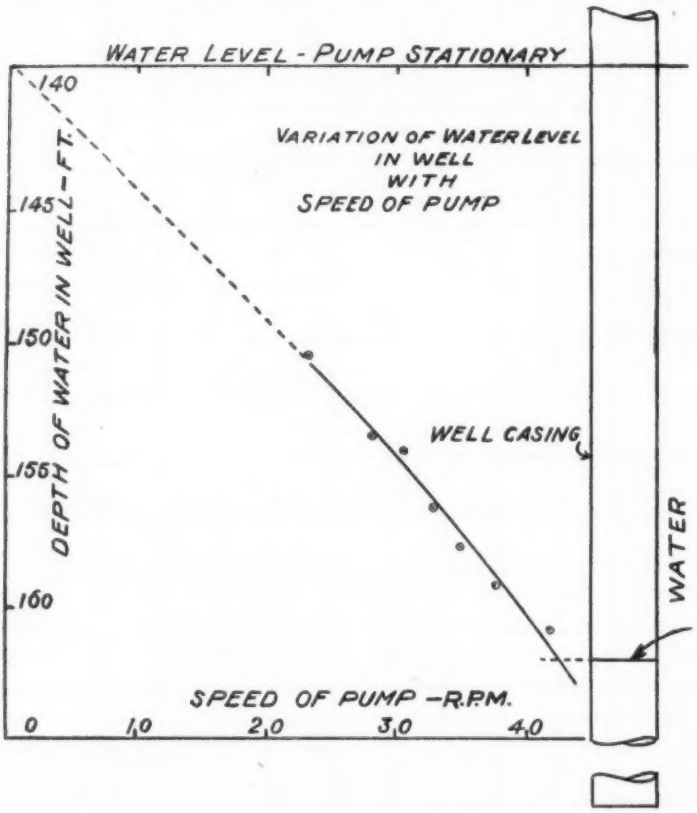


FIG. 4. EFFECT ON WATER LEVEL OF SPEED OF PUMP

evident that a more detailed study of one pumping unit might yield the characteristics desired. Accordingly, Well 10 was chosen for these tests, principally on account of the ease with which the water level in it could be measured with the pump in operation, and also because the pump in this well was in average mechanical condition, having been in service about five years.

TABLE 2

Characteristics of pumping unit, varying speed (10 horse power motor)

MOTOR PULLEY	R. P. M. PUMP	MOTOR K. W.	INTAKE H. P.	MOTOR		PUMP INTAKE H. P.	GAL./MIN.	HEAD	OUTPUT H. P.	PUMP EFF.	OVER-ALL EFF.	K. W. H. PER 1000 GAL.	GAL. PER MIN. NOMINAL	NOMINAL SLIP	
				Eff.	P. F.									Gal./min.	Per cent
inches															
4	22.25	3.75	5.03	85.5	61.0	4.3	57.0	150.75	2.22	51.7	44.2	1.096	75.2	18.2	24.2
5	27.25	4.64	6.22	86.0	68.0	5.35	74.0	153.75	2.88	53.8	46.3	1.045	92.0	18.0	19.7
5½	29.5	5.05	6.77	86.5	70.5	5.85	81.8	154.33	3.20	54.7	47.3	1.028	99.8	18.0	18.0
6	31.75	5.42	7.26	86.5	73.0	6.28	89.0	156.5	3.52	56.0	48.5	1.014	107.2	18.2	17.0
6½	33.75	5.78	7.75	87.0	75.0	6.75	95.7	158.0	3.81	56.5	49.2	1.005	114.0	18.2	16.1
7	36.5	6.25	8.37	87.0	77.0	7.29	105.0	159.5	4.24	58.2	50.7	0.992	123.2	18.2	14.8
8	41.0	7.08	9.48	87.0	79.0	8.25	120.0	162.25	4.92	59.6	51.8	0.984	138.4	18.4	13.3

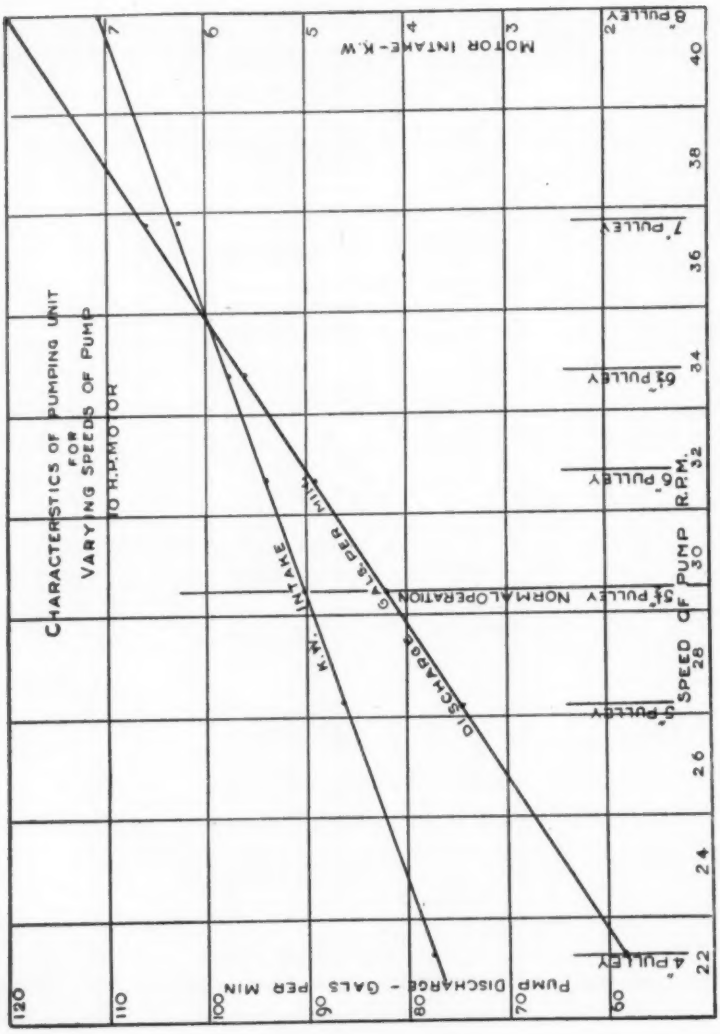


FIG. 5. EFFECT OF SPEED OF PUMP ON DISCHARGE AND POWER CONSUMPTION

In order to get at the pumping unit characteristics for a variation of speed of pump and water discharged, a number of pulleys were arranged for use on the driving motor, of such sizes that pump speeds of from 22 to 41 r.p.m. were obtained. The results of this test are shown in Table 2. Figure 4 shows the change of the water level in the well as the pump speed was increased. It may be observed that this curve bends down at the higher speeds, that is,

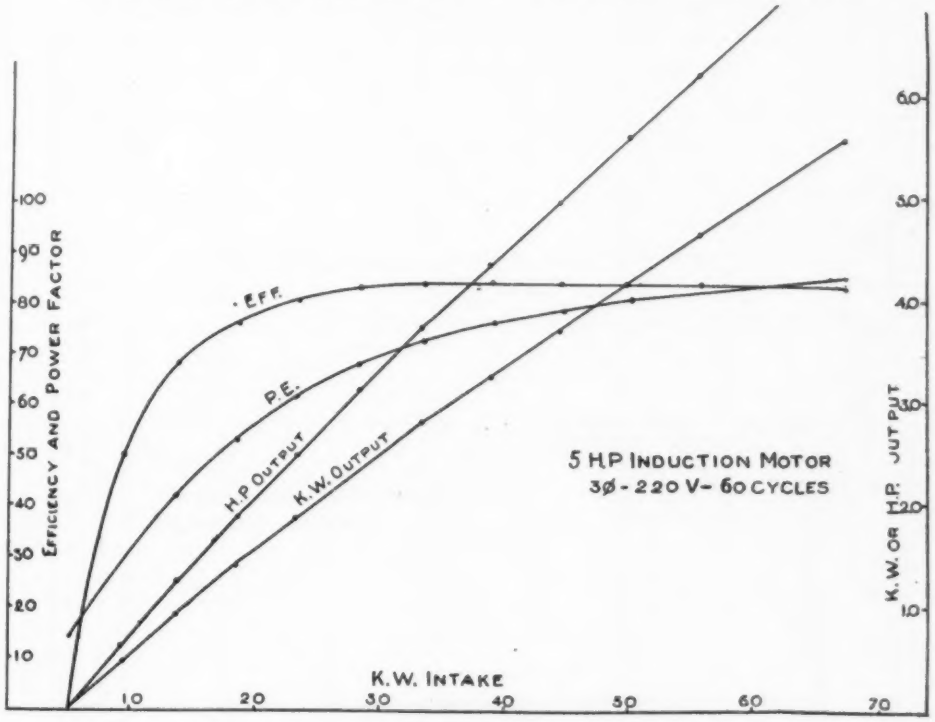


FIG. 6. CHARACTERISTICS OF 5 HORSE POWER MOTOR

the water level goes down faster as the discharge of the pump increases.

Other original data taken for varying speeds of pump are shown in Figure 5, from which it is seen that both the power required by the motor and the water discharged from the pump are linear functions of the speed, within the range taken. Due to the changing head against which the water is pumped, under these conditions, and the change in efficiency of the pumping unit, this direct relation

would probably not hold for wider variations in speed. In this case, however, the increase in efficiency at higher speeds appears to just about offset the increased head through which the water must be lifted.

Figures 6 and 7 show the electrical characteristics of 5 and 10 horse power induction motors used with these wells. These curves are here plotted against kilowatt intake of the motor, since this is

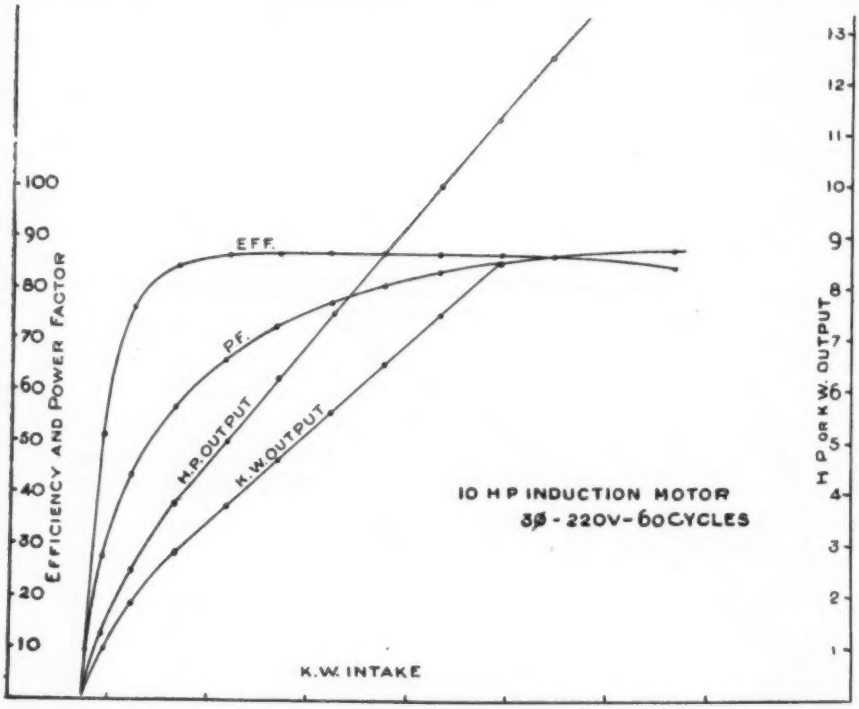


FIG. 7. CHARACTERISTICS OF 10 HORSE POWER MOTOR

the readily measurable quantity. It may be observed that the efficiency of both motors remains fairly constant over a wide range of loads, and that the 10 horse power motor efficiency is slightly higher than that of the 5 horse power motor. In the matter of power factor, however, it is evident that this may be very low if the motor is underloaded. Although low power factor does not lower the efficiency of the motor or pumping unit, it has a very serious effect upon the generating plant. It may result in drawing full

load current from the electric generator although the actual power delivered is far less than normal. A further increase of load on the system, then, will require additional generating capacity in the plant with greatly increased investment and operating expense, and will result in only partially loaded prime movers. Even if the electrical energy for operating the wells is purchased, there is a tendency, at the present time, to penalize low power factor in rate schedules

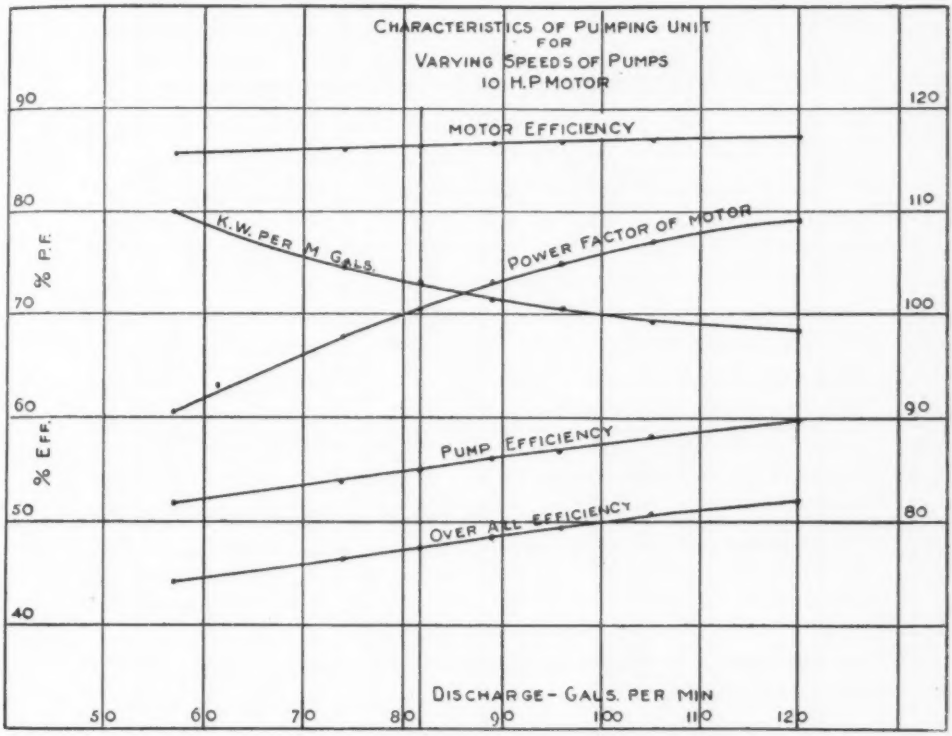


FIG. 8. EFFECT OF SPEED VARIATIONS ON CHARATERISTICS OF 10 HORSE POWER PUMP

for large customers. It is of first importance, then, to select motors of such size that the power factor of the system will be reasonably high, or to resort to the use of a synchronous motor for power factor correction.

Figure 8 shows the change in power factor of the 10 horse power motor as the discharge is increased (by speeding up pump) from about 57 gallons per minute to 120 gallons. The power factor

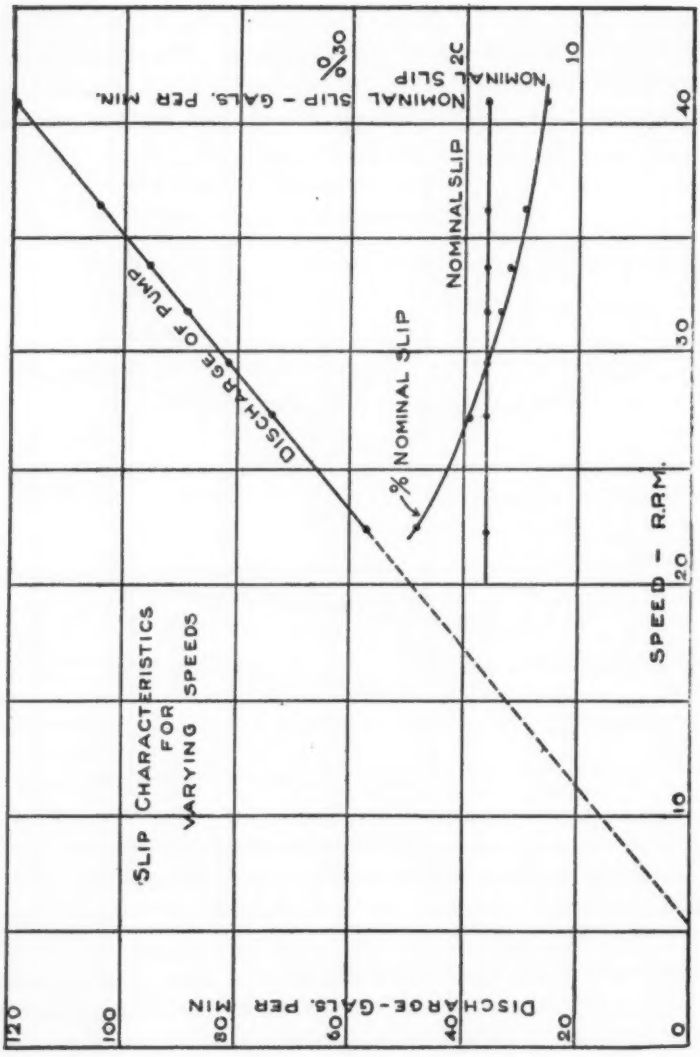


FIG. 9. SLIP OF PUMP AT DIFFERENT SPEEDS

thus changes from a very poor one, at light load, to a reasonably good one at high speed. The motor efficiency increases but little through this range of loads, but the pump efficiency increases to a marked extent and the over-all efficiency curve is practically parallel to it. This increased efficiency at high speeds is shown from another point of view by means of the curve of economy, kilowatt-hours per 1000 gallons pumped.

In order to explain this increased efficiency at high pumping speeds, curves of slip are shown in Figure 9. It is seen that, with a given column of water in the pump barrel and discharge pipe, the water will leak past the pistons and valves at the same rate whatever the speed of the pump. The nominal slip is here used, i.e., the slip calculated on the assumption that the effect of overlapping travel of the pistons is zero. The percentage of slip, then, decreases with increased speed and thus the efficiency rises for high discharge rates. The friction losses in the pump head should increase in direct proportion to the speed, affecting the pump efficiency but little.

It is well known that the speed of a reciprocating pump cannot be increased indefinitely on account of the increased wear and tear involved. But, in a well designed pump, one which runs smoothly and with minimum shock, it is probable that speeds might be raised to advantage even at the expense of increased maintenance. For instance, if three wells are in operation and more water is needed it might be economical to increase the speed of the three pumps and pay a possibly increased operating expense, rather than increase the investment by driving and equipping a new well. The increased efficiency of the pumps operating at higher speed also makes the three well proposition look more favorable.

In conclusion, the authors feel that the study of motor-operated deep-well pumps is worthy of considerable experimental investigation, and the present paper is only a preliminary report on the problem at hand.

RUSTING OF PIPE IN SERVICE[1]

By Harry Y. Carson

The history of pipe is the history of civilization. Upon no other single product have the great cities of the world depended in such large measure for their health and comfort. The modern city is dependent upon pipe as we are dependent upon arteries, veins and capillaries in human bodies. Let us compare our modern buildings to the individual cells in our body. These cells receive their life-renewing supply of wholesome water through multitudinous pipes, permeating every wall and corridor. After the water has supplied life, bathed and cleansed to its utmost, it carries off the waste products through other capillaries, collecting into larger and still larger main trunk arteries. We are taught that man has a natural life of three score and ten years, but would he not have a life of only five years if his capillaries were constructed to last but that long? If his capillaries were built on the century basis and his arteries on the ten year basis what would be the ultimate result?

Cast iron and wrought pipe compared. Pipe made in a rolling mill does not resist corrosion in service so well as pipe made in a foundry. Both kinds of pipe may have been made from the same quality of iron ore: both are smelted and both may have been treated in the same manner up to a certain point in their refinement known as the "pig iron" stage. The iron which passes though a tube mill must first of all be rendered comparatively free from carbon, silicon, sulphur, phosphorus and manganese before it can be manipulated into thinly rolled sheets for the purpose of making wrought pipe. Paradoxical as it may seem to the layman's mind, the question of chemical refinement plays small part in comparison to other factors. These factors which make for rust resistance of metal pipe have heretofore been given little consideration in the many discussions that have taken

[1] Read before the New York Section at New York on February 21, 1917.

place on this subject. Pipe or fittings made of cast steel last quite as well as ordinary gray cast iron in service, but conversely the pipe which may have been rolled out from a cast steel ingot lasts not one-tenth as long as the cast product. Now why should this be so?

Perhaps further light on this very involved question can be found in a recent investigation by a prominent tin plate manufacturer, who tried to ascertain the reason why the old iron links of the suspension bridge at Newburyport, Mass., have so successfully withstood the elements of wind and rain for more than one hundred years, little attention having been given to painting or protecting the metal. In reference to this the following statement is made in *The Corrosion of Iron*, a book published in 1916 by L. C. Wilson:

> To explain how or why these (links) have so successfully withstood the attacks of air and moisture for so many years is not as easy as it might seem for apparently some of our commonly accepted notions and theories as to what a good iron should and should not be do not hold, or at least are offset by other conditions. For instance, some of the links of the Newburyport bridge have been examined and found to be of very ordinary purity and of a most heterogeneous structure, segregation of the impurities being decidedly noticeable. The question of any inherent superiority in the iron itself was settled by heating and rolling one of the links into a sheet and exposing it to the weather, when it was found that it corroded quite as readily and extensively as the rolling mill product of today.

Structure of cast iron and rolled iron. At the top of the chart, Figure 1, which is a drawing reproduced from a standard text book on metallography, there are indicated the various commercial grades of iron products which are familiar to all. Beginning with the pig iron on the extreme right it will be noted that the carbon content is greater than 4 per cent. Next comes gray iron with a carbon content of 3.2 to 4.6 per cent. White iron is next with a carbon content of 2.2 per cent to 3.2 per cent. The iron which has a carbon content less than 2 per cent is known as steel, and this material may be rolled or manipulated while all other classes of iron must be cast into the desired shapes or objects. Glancing at the extreme left hand side of the chart, the various temperatures at which iron melts and freezes or solidifies may be observed. These temperatures are somewhat lower for gray iron than for steel, and are yet higher for pig iron than for gray iron. However, these very high temperatures, above 2200°F., apply only to the cast irons and cast steels, for when

we begin to investigate the temperatures at which steel pipe and wrought iron pipe are made we must confine our attention to that part of the chart below 1600°F. and between the limits 0 and 0.1 per cent carbon. This metal is worked at more than 1000°F. below its freezing temperature and hence does not attain its full crystal growth which can be restored to the internal structure of the pipe only by remelting.

As a result the pipe so made displays a form of flaking or exfoliation of its particles under the corrosive influence of ordinary service conditions. A very good analogy to iron rusting can be found in the geological structure of various grades of stone found on the

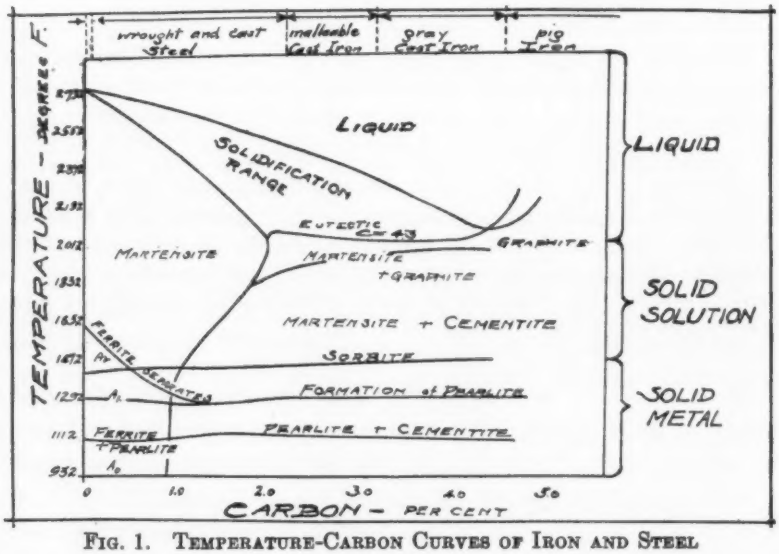


FIG. 1. TEMPERATURE-CARBON CURVES OF IRON AND STEEL

earth's surface. Those stones, such as granite, which were molded or "cast" ages ago at a very high temperature are far more durable than such materials as sandstone which were laid down and cemented together after the earth's surface had cooled to a low temperature. Both kinds of stone may be equally pure in silica and, moreover, we may make from sandstone as sturdy and durable a product as granite, if we but supply a sufficient amount of heat to fuse together the aggregate mass of silica crystals. Figure 2 is a photomicrograph of cast iron showing the bonding of the crystals.

Contrary to what has so often been stated with regard to cast iron

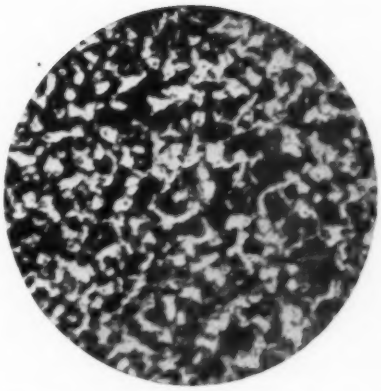


FIG. 2. PHOTOMICROGRAPH OF CAST IRON

FIG. 3. PHOTOMICROGRAPH OF STEEL (0.08% CARBON)

FIG. 4. RUST FORMATION ON CAST IRON

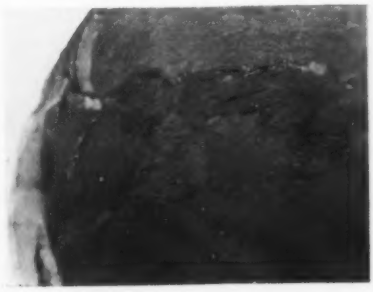


FIG. 5. RUST FORMATION ON WROUGHT IRON

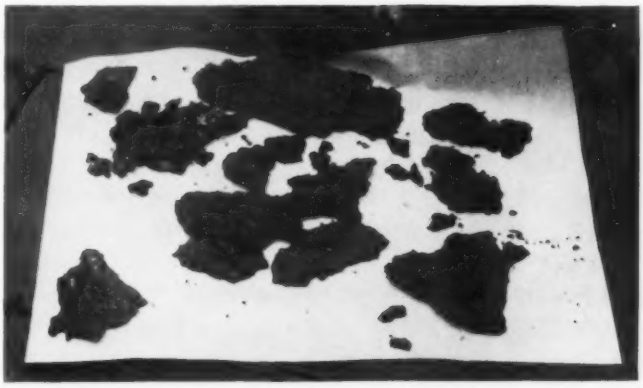


FIG. 6. RUST FLAKES FROM WROUGHT IRON AND STEEL PIPE

owing its life altogether to a "skin" resulting from the casting process, the author must diametrically disagree for the simple reason that castings have been in service for more than two hundred and fifty years and have shown no greater loss wherever the original casting skin has been totally absent.

The effect of rolling and of re-rolling is even more noticeable under the microscope than is the effect upon the internal structure of rolled steel. A bar of metal which is rolled or drawn "cold" will, if the process be carried to the extreme, present a structure viewed parallel to the direction of drawing, like the photo-micrograph shown in Figure 3. However, the pipe made from wrought iron is made at a final heat treatment of approximately 1200°F. which gives a fair degree of fusion to the gross slag particles distributed through the skelp from which the pipe is made. The inhibiting properties of this slag when carefully distributed through the wall section of the pipe compensates for the low temperature at which wrought iron pipe is rolled. Nevertheless both wrought iron and steel pipe last approximately the same under like conditions of service.

The author believes that the extraordinary qualities of cast iron pipe and of cast iron fittings can best be explained on the basis of heat treatment. At least it is the only explanation that seems to coincide with all of the observed corrosion phenomena which are incidents in the rusting of pipe in service.

Theories of corrosion. The many experimenters who have investigated this subject have evolved at least three different theories of iron pipe corrosion.

The hydrogen-peroxide theory explains the phenomena of rusting by assuming that when iron, water, and oxygen are in contact a chemical reaction takes place in which ferrous oxide, FeO, and hydrogen peroxide, H_2O_2, are formed. Then the ferrous oxide is supposed to react with a portion of the hydrogen peroxide to form a basic or hydrated oxide; rust, an insoluble basic oxide, is the final product.

A second theory, known as the carbonic acid theory, holds that rusting is always started by an acid. It assumes that even a very weak carbonic acid will first convert a portion of the iron into a ferrous salt, with the escape of hydrogen. Under the influence of oxygen and water the ferrous salt is oxidized to ferric hydroxide $2Fe(OH)_3$. The ferric hydroxide is insoluble and upon precipitation the original carbonic acid is liberated. The carbonic acid is then free to attack more iron and to repeat the process of reducing the iron to rust.

The third and perhaps most important theory of iron corrosion is known as the electrolytic theory. It rests on the assumption that when water comes into contact with an iron surface a certain amount of the iron goes into solution. This small amount of iron carried in solution in the water produces an electrolyte. The process of rusting is then explained by the familiar galvanic electrolysis which takes place between the points of high and low potential on the surface of the metal. The modern conception of the reactions which take place holds that they are accompanied by certain readjustments of the electrical state of the reacting ions and the iron must go into solution as a ferrous ion before it becomes oxidized.

If it were possible to summarize all of the painstaking research that has been carried on by investigators since 1800, we would probably find that one common point has always been mutually agreed upon. It has always been admitted that the iron particles must be in direct contact with oxygen and moisture before rusting can take place. In all likelihood, it is probably close to the truth to say that under varying conditions of service for iron pipe, any one or all of the corrosion theories actually apply during the process of rusting. As for inhibiting or preventing the rusting of an iron pipe line, the problems, both theoretical and practical, seem to sum up in the one proposition of keeping moisture and oxygen from reaching the iron itself. Expressed in different terms it is the old question of getting a protective coating on all exposed parts of the pipe.

Protective skin on cast iron. There is an entirely different kind of surface rust formation on cast iron pipe than is to be found on the surface of a pipe made of rolled iron or steel. Figure 4 shows the rust formation on cast iron and Figure 5 that on wrought iron pipe after they have been in the same service for 18 years. On the surface of a cast iron pipe, whether machined or fractured or not, may be observed a reddish-brown colored iron oxide which is very hard, and so tenaciously covers the surface of the pipe that it penetrates down between every surface crystal and forms a perfectly impervious coating which is not penetrated by the moisture and oxygen. Hence galvanic action stops on such a surface and the life of the pipe is insured by the very virtue of the oxide itself contained on the surface.

On the other hand let us observe the surface of a piece of wrought iron pipe or a piece of steel pipe. In Figure 6 we see a loose scaling formation of rust which is soft and pervious to moisture and oxygen. These flakes of rust are very readily removed with a pen-knife or

the thumb nail from the surface of the pipe. Once the rust scale drops away the fresh bright particles of the iron are exposed to the oxygen and moisture so that further rusting takes place on each fresh surface until the entire wall thickness of the pipe has been destroyed. The surface scales on wrought iron materials accelerate galvanic action, whereas the rust formation on cast iron has just the reverse quality, namely, the prevention of galvanic electrolysis. The cause for the gradual slowing up of rust action on cast iron seems, therefore, to be thoroughly explained by the quality of the surface oxide.

Plumbing and wasted water. The part which bad plumbing plays in the enormous waste of public water is a very significant reason why water works officials should pay more attention to the details of the piping installed in the buildings. The habit has been too common of thinking that the responsibility ends at the corporation cock on the street main or at the meter. That responsibility should not end short of the spigot on each plumbing fixture.

The New England Water Works Association received last year an excellent report on service pipes in which the following table was given to show the comparative life of some materials which have been used:

	YEARS BEFORE TROUBLE BEGINS	LIFE OF PIPE YEARS
Plain iron or steel pipe	12	16
Galvanized pipe	15	20
Lead pipe	10	35
Lead lined pipe	10	23
Cement lined pipe	14	28

These data are valuable as giving some idea of the amount of water wasted from steel and wrought iron service pipes, which wastes through rust holes in the pipe long before the trouble is detected. Moreover, it becomes difficult to learn the amount of wastage until the line is torn up and replaced. The table also shows that galvanized pipe is little better than black pipe.

Cast iron pipe for service lines in the streets as well as for the mains has been installed at low cost, and perfect results can be obtained provided satisfactory joints are used. The minimum size should be 2 inch internal diameter and this can be laid at approximately the same cost per foot as 1 inch lead service pipes. Nothing

will ever reduce the economic loss from wasted water, amounting to $4,000,000 in the United States annually, so much as the general adoption of cast iron pipe with joints capable of remaining permanently tight. Such piping should begin at the pumping station and continue as far as the plumbing fixtures in the buildings. For pipes less than 2 inches internal diameter the material should consist of brass or copper. The general rule should be to use 2-inch cast iron service pipes, but it is not necessary to provide more than a ¾- or 1-inch tapping on the main. The meter may also be of 1-inch size on a 2-inch service pipe without causing any appreciable pressure loss due to friction.

In connecting the service pipe to the street main the present practice of using a short lead gooseneck between the cast iron pipe and the brass corporation cock should be followed in order to provide for settlement and expansion. The service piping must have joints which are flexible as well as tight against pressure. Such a joint is on the market.

The bell and spigot ends of the pipe are machined at slightly different tapers and bolts are used for assembling the joint. When the joints have been made tight against pressures the bolts are usually loosened off and this construction remains permanently tight as there is no chance of deterioration either at the joint or at any part of the pipe system. The Underwriters' Laboratories have tested and approved for high-pressure fire service the cast iron piping having this joint.

SOCIETY AFFAIRS

ILLINOIS SECTION

The fifth meeting of the Illinois Section, corresponding to the twelfth meeting of the Illinois Water Supply Association, was held at the University of Illinois, Urbana, March 13–14, 1917. Prof. C. R. Richards, Dean of the College of Engineering of the University of Illinois, welcomed the members to the University.

President Paul Hansen delivered the presidential address on the subject, "What the Illinois Section has done and can do." The Secretary's report showed that there are now 137 active members residing within the State of Illinois. This is a decided increase over the 117 in June, 1915, and 55 in 1913, before the Section was formed. The normal is not yet reached, for at the time of the last convention of the Illinois Water Supply Association it had 162 active members residing in Illinois. It is perhaps more difficult to persuade new members to join now that it is necessary to pay an initiation fee of five dollars. It is hoped, that the increase in membership may continue. The Second Illinois Drainage Conference was held on March 13, 14, and 15. There was no attempt to have joint sessions, with the exception of the annual banquet. Members of the two organizations were left free to attend the meetings of either organization.

The Treasurer's report showed that the expenses of the Section had been held well within the limits allowed by the general society. The statement concerning the expenses will be found in the report of the Treasurer of the general society.

Invitations to hold meetings at Springfield, Peoria, and Evanston were received, but the place of holding the next meeting was left to the Executive Committee.

Resolutions were adopted expressing the sympathy of the Section to one of the principal speakers, Edmund T. Perkins, because of the recent death of his wife. The following letter was sent to Mr. Perkins:

The members of the Illinois Section of the American Water Works Association at the ninth annual meeting held on March 13, have learned with deep concern of your recent bereavement and have instructed the undersigned as a committee to convey to you the sympathy of the members individually and collectively in your great loss.

F. H. NEWELL,
JOHN W. ALVORD,
JACOB A. HARMON.

Papers were presented at the five sessions of the meeting as follows:

"Operation of Municipal Combined Electric Power and Waterworks," W. J. Spaulding. Discussion by Kohn, Hansen, Bartow, and March.

"Seeking Better Insurance Classification for Small Cities," C. M. Roos. Discussion by Ringness, Spaulding, and Kennicott.

"Experience with Meters at Elgin," C. L. Kohn. Round-table discussion "Meter-department Operation," Ringness, Craven, Spaulding, Lautz, and Brower.

"Some Costs of Maintenance of Motor-driven Deep-well Pumps," M. L. Enger. Discussion by Maury, Havermeyer, and Strasser.

"Ultra-violet Rays, Advantages and Disadvantages in the Purification of Drinking Water" (illustrated), Dr. R. R. Spencer, Assistant Surgeon, United States Public Health Service, Chicago. Discussion by Jennings, Smith, and Bartow.

"Water Supply as Affected by Regulation," W. A. Shaw.

"Relation of the Railroad to the City Waterworks Plant," C. R. Knowles. Discussion by Leisen, Hansen, Spaulding, MacDonald, and Ely.

Round-table discussion "The Value and Relation of Laboratory Control to the Operation of Water-purification Plants," Gelston, Fritze, Dutton, Ely, and O. M. Smith.

"Public Utilities and Public Health," H. E. Barnard, State Food and Drug Commissioner, Indianapolis, Indiana. Discussion by Spaulding, Bartow, Henderson, Hansen, Leisen, and Strasser.

"A Municipal Waterworks Constructed and Operated Solely for Industrial Service," C. L. James (by title).

"Proposed Reorganization of the Illinois State Board of Health," Dr. C. St. Clair Drake (read in his absence by Paul Hansen). Discussion by Bartow, Hansen, Alvord, Spaulding, Kohn, Greeley, and Craven.

"Flow Measurement in a Collecting Gallery by the Chemical Method," Donald H. Maxwell. Discussion by Alvord, Hansen, Bartow, and Spaulding.

"Sterilization by Liquid Chlorine and Hypochlorite of Lime," M. S. Dutton.

"Some Tests on Motor-driven Deep-well Pumps," Ira S. Fisk and P. S. Biegler, Electrical Engineering Department, University of Illinois.

"Treatment of Public Water Supplies from Lake Michigan," S. A. Greeley. Discussion by Brower.

"Increasing a Water Supply," (illustrated), H. M. Ely.

"Some Typical Railway Waterworks Installations," (illustrated), C. R. Knowles.

"Illustrations of the Denver Union Water Company's Property," W. W. DeBerard. (A Supplement to the paper presented in 1915; see Vol. 2, p. 442.)

Luncheon was served each day at the University Club. Visits were made to the new laboratories of the State Water Survey, the newly installed pressure filters at the swimming pool in the Men's Gymnasium, the new deep well constructed for the University of Illinois by Layne and Bowler, the pumping station and iron-removal plant of the Champaign and Urbana Water Company, and the sewage-experiment station where experiments with activated sludge are being carried on by the State Water Survey.

At the annual dinner, which was held with the Second Annual Illinois Drainage Conference, Theodore Leisen, General Superintendent of the Board of Water Commissioners, Detroit, Michigan, delivered the principal address. Owing to unavoidable absence of the other speakers, Edmund T. Perkins, who was unable to be present because of the recent death of his wife, and Dr. Edmund J. James, President of the University of Illinois, who was detained by an important meeting of the University trustees, Capt. B. W. Benedict, a member of the faculty of the University of Illinois, gave an interesting description of the experiences of the University Battery on the Mexican border during the summer, and the toastmaster, Paul Hansen, called for a large number of three-minute speeches.

The members attended the review of the University brigade of 2400 men. The brigade has two regiments, two full bands, engineer corps, hospital corps, signal corps, and a regular battery of artillery.

Officers for the ensuing year were elected as follows: Chairman:

E. MacDonald, Superintendent Water Company, Lincoln; Vice-Chairman: W. W. DeBerard, Western Editor, *Engineering News-Record;* Treasurer: H. E. Keeler, President, Rogers Park Water Company; Trustee: L. A. Fritze, Chemist and Superintendent of Filtration, Moline Water Department, Moline. The hold-over Trustees are F. C. Amsbary and W. E. Lautz. Edward Bartow was reëlected Secretary.

NEW YORK SECTION

A meeting was held at Hotel Astor, New York, on February 21, 1917, at which 99 members and guests were present. The section chairman, Allen Hazen, presided. Brief statements on the Association's affairs were made by President Metcalf and Secretary Diven. The secretary of the section, Wm. W. Brush, submitted a report showing an increase of 40 in the membership of the section during the year; the section now has 258 active members, 45 associate members and 14 corporate members. The total receipts during the year were $337.30, of which the Association contributed $59.20, and the expenditures were $329.20. George A. Johnson was elected a member of the governing board for five years, to succeed Robert E. Milligan. The officers of the section are: Chairman, Allen Hazen; Secretary and Treasurer, Wm. W. Brush; Governing Committee, Allen Hazen, Henry deF. Baldwin, Morris R. Sherrerd, Wm. W. Brush and George A. Johnson.

A paper on "Service Pipes" was presented by F. N. Speller and discussed by W. P. Mason, R. E. Milligan and the author. It was illustrated by lantern slides and moving pictures. A paper on "The Rusting of Pipe in Service," illustrated by lantern slides, was presented by Harry Y. Carson.

FOUR STATE SECTION

The annual meeting was held at the Bellevue-Stratford Hotel, Philadelphia, on April 18, 1917, with the chairman of the Section, Carlston E. Davis, in the chair. In the absence of Mayor Smith, the members and guests were welcomed by John C. Trautwine, Jr. Mayor Ellis, of Camden, N. J., made an address on Water-Works Preparedness. A paper on "The Service Charge; How Constituted," was read by J. N. Chester and brought out an unusually animated discussion.

J. W. Ledoux was re-elected a member of the executive committee. The officers of the section for the current year are: Chairman, Carleton E. Davis; Vice-Chairmen, E. M. Hoopes, A. M. Quick and L. Van Gilder; Secretary-Treasurer, Chas. R. Wood; Executive Committee, J. W. Ledoux, N. E. Bartlett, Wm. H. Boardman, John C. Trautwine, Jr., and Henry D. Brown.

OBITUARY

William H. Bloom, Williamsport, Pa., died January, 1917.
Lyman E. Cooley, Chicago, Ill., died February 10, 1917.
Robert M. Kellogg, Brooklyn, N. Y., died April 1, 1917.
Alexander MacCullum, Philadelphia, Pa., died February 3, 1917.
W. F. McCue, Denver, Colo., died December 27, 1916.

NEW MEMBERS

Active

Allen, Hugh L., Superintendent, Water and Light Commission, Greenville, N. C.

Andrews, George C., Water Commissioner, Buffalo, N. Y.

Barr, Dr. William M., Consulting Chemist, Omaha, Neb.

Bernhagen, Lewis O., Chemist and Superintendent of Filtration, Minneapolis, Minn.

Blomquist, H. F., City Engineer, Mankato, Minn.

Bruce, T. E., Superintendent Water Company, Tupper Lake, N. Y.

Childs, J. A., Engineer, State Board of Health, St. Paul, Minn.

Chipley, Dudley, Engineer and Superintendent of Water Works, Columbus, Ohio.

Darrow, George P., Accountant, Bureau of Water, Philadelphia, Pa.

Day, Leonard A., Chief Engineer, Operating Section, Water Department, St. Louis, Mo.

Harding, C. T., Superintendent of Water and Light Works, Virginia, Minn.

Harmon, Jacob A., Consulting Engineer, Peoria, Ill.

Johnston, William J., Superintendent Water Works, Marquette, Mich.

Keliher, Timothy, Superintendent Water Company, Williamsport, Pa.

Kirkpatrick, Earl T., Chief Chemist and Bacteriologist, Des Moines, Iowa.

Lee, H. R., City Chemist, Decatur, Ill.

Maxwell, Donald H., Assistant Engineer with Alvord & Burdick, Chicago, Ill.

Mohler, B. M., Chemist, State Board of Health, Minneapolis, Minn.

Newell, C. W., Service Foreman of Maintenance, Minneapolis Water Department, Minneapolis, Minn.

Newell, F. H., Consulting Engineer, Urbana, Ill.

Newhall, William G., Assistant Superintendent, Portland Water District, Portland, Me.

O'Meara, Robert J., Assistant Engineer, Department of Water Supply, New York, N. Y.

Pedroso, Dr. Alex. M., Sao Paulo, Brazil.

Perrow, Dr. M. G., Lynchburg, Va.

Powers, Jerome, Civil Engineer, Elizabeth, N. J.

Rowan, J. J., Superintendent Water and Light Works, Hibbing, Minn.

VanDeusen, W. P., Assistant Chemist, Water Department, Minneapolis, Minn.

Wallace, Robert B., Water Works Trustee, Council Bluffs, Iowa.

Wilhelm, E. G., Secretary and Treasurer, Williamsport Water Company, Williamsport, Pa.

Corporate

Marion Water Company, Marion, Ohio.

Sidney Water Works Company, Sidney, Ohio.

Associate

Maritimo Coating Corporation, 1733 Grand Central Terminal, New York, N. Y.

Schramm and Son, Chris. D., 308 North Fourth Street, Philadelphia, Pa.

Stewarts and Lloyds, 41 Oswald Street, Glasgow, Scotland.

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July 1, 1916

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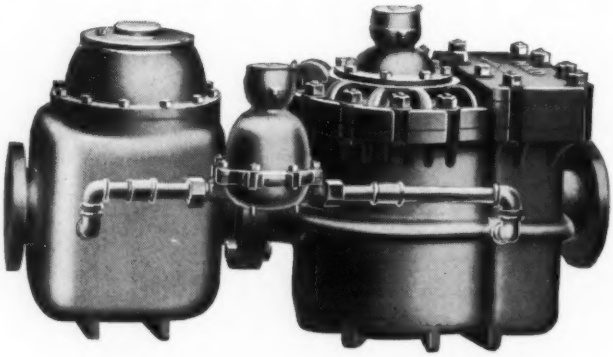

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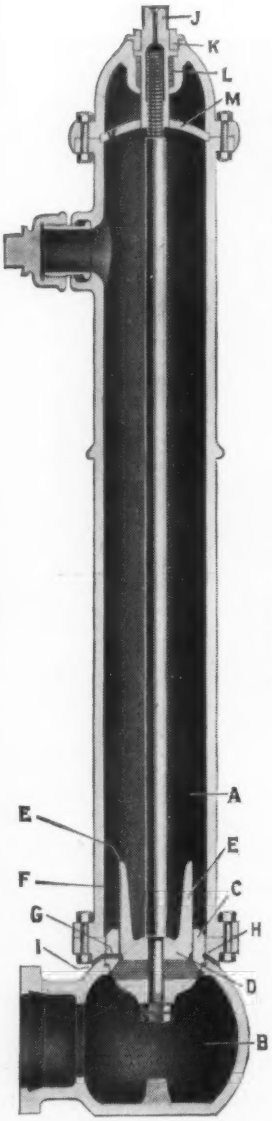

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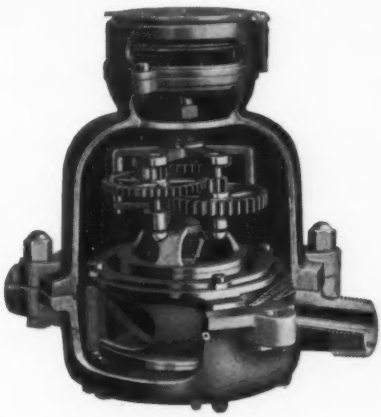

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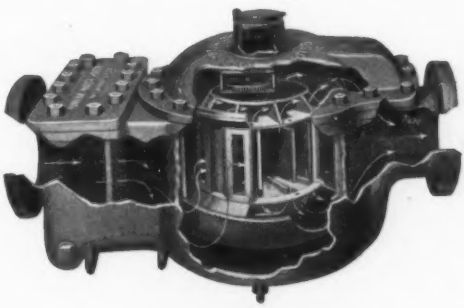

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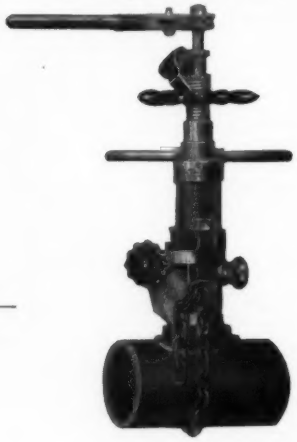

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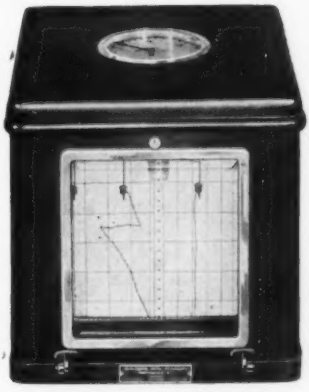

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